SOUTH FLORIDA
WATER MANAGEMENT DISTRICT

EAA Storage Reservoir A-1
Basis of Design Report

January 2006

ACCELER8
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ABBREVIATIONS AND ACRONYMS

A	Amps
AASHTO	American Associate of State Highway Transportation Officials
ACES	Automated Coastal Engineering System
ACHP	Advisory Council on Historic Preservation
ACI	American Concrete Institute
ADAAG	Americans with Disabilities Act Accessibility Guidelines
AFB	USACE's Alternate Formulation Briefing
AGMA	American Gear Manufacturers Association
AISC	American Institute of Steel Construction, Inc.
ANSI	American National Standards Institute
AOR	Allowable Operating Region
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating and Air-Conditioning Engineers
ASME	American Society of Mechanical Engineers
AST	Aboveground Storage Tanks
ASTM	American Society for Testing Materials
ATS	Automatic Transfer Switch
AWWA	American Water Works Association
B&V	Black & Veatch Corporation
BACT	Best Available Control Technology
BEP	Best Efficiency Point
bgs	below ground surface
BHRA	British Hydraulics Research Association
BODR	Basis of Design Report
BTU	British Thermal Unit
caprock	near surface limestone layer, typically EL 2.6 to 6.6 (NAVD)
CAA	Clean Air Act
CCM	Criteria Committee Meeting
CCTV	Closed Circuit Television
CE	Categorical Exemption
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CERP	Central Everglades Restoration Project
CERPRA	Comprehensive Everglades Restoration Plan Regulation Act
cfm	cubic feet per minute
cfs	cubic feet per second
CMU	Concrete Masonry Unit
COC	Constituents of Concern
CPHU	County Public Health Unit
CSM	cfs per square mile
CWA	Clean Water Act
CZMA	Coastal Zone Management Act

DCM	Design Criteria Memorandum
DEP	Florida Department of Environmental Protection
DMSTA	Dynamic Model for Stormwater Treatment Areas
DMSTA2	Dynamic Model for Stormwater Treatment Areas Version 2, June 2005
DOH	Florida Department of Health
DSS	Decision Support System
EAA	Everglades Agricultural Area
EAA SR	Everglades Agricultural Area Storage Reservoir
EAP	Emergency Action Plan
ECM	Electronic Control Module
ECP	Everglades Construction Project
EFA	Everglades Forever Act
EFH	Essential Fish Habitat
EIS	Environmental Impact Statement
EM	USACE Engineering Manual
EPA	United States Environmental Protection Agency
EPD	EAA Environmental Protection District
ERM	Environmental Resource Management
ESA	Endangered Species Act
ET	Evapotranspiration
F.A.C.	Florida Administrative Code
f’c	concrete reduction factor
FDACS	Florida Department of Agriculture and Consumer Services
FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FIU	Field Interface Units
FLUCCS	Florida Land Use, Cover and Forms Classification System
FNAI	Florida Natural Areas Inventory
FoS	Factors of Safety
FPL	Florida Power & Light
Fpm	feet per minute
fps	feet per second
FS	Florida Statutes
FSI	Formed Suction Intake
FFWC	Florida Fish & Wildlife Conservation Commission
fy	Steel Yield Strength
gal	gallons
GCP	Generic Construction Permit
GFCI	Ground Fault Circuit Interrupter
GIS	Geographical Information Systems
GP	General Permit
gpm	gallons per minute
HI	Hydraulic Institute
HMR	hydrometeorlogical reports

Hp	horsepower
HVAC	Heating, Ventilation, and Air Conditioning
HW	Headwater
IDF	Inflow Design Storm
IMC	Interagency Modeling Center
I/O	Input/Output
ISAFP	Initial Storage Filling Plan
K_h	Horizontal hydraulic conductivity
K_v	Vertical hydraulic conductivity
kV	Kilovolts
kW	Kilowatt
KVA	kilovolt-ampere
LCC	Life Cycle Cost Analysis
LED	Light Emitting diode
LOPA	Lake Okeechobee Protection Act
mA	milliamps
MCC	Motor Control Center
mgd	million gallons per day
MGM	million gallons per month
mm	millimeter
mph	miles per hour
MSL	Moon sea level
MSS	Manufacturers Standardization Society
mV	millivolt
MWL	Maximum Water Level
NAAQS	National Ambient Air Quality Standards
NAD83	North American Datum of 1983
NAVD	North American Vertical Datum of 1988
NAVD88	North American Vertical Datum of 1988
NC	Not Calculated
NEMA	National Electrical Manufacturers Association
NEPA	National Environmental Policy Act
NETM	National Environment Technical Memorandum
NFPA	National Fire Protection Association
NFSL	Normal Full Storage Level
NHC	National Hurricane Center
NHPA	National Historic Preservation Act
NNRC	North New River Canal
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NPSH	Net Positive Suction Head
NPSHA	Net Positive Suction Head Available
NPSHR	Net Positive Suction Head Required
NRCS	United States Natural Resources Conservation Service
NSR	New Source Review
NVGD	National Geodetic Vertical Datum of 1929

NVGD29	National Geodetic Vertical Datum of 1929
NWL	Normal Water Level
O&M	Operations and Maintenance
O.D.	Outside diameter
ODP	Open, Drip Proof
OG	Original Grade
OoM	South Florida Water Management District's Office of Modeling
OSHA	Occupational Safety & Health Administration
PAL	Planning Aid Letter (PAL)
PBC	Palm Beach County
PFI	Pipe Fabrication Institute
PIR	Project Implementation Report
PLC	Programmable Logic Controller
plf	pounds per linear foot
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
POM	Project Operating Manual
POR	Period of Record
POR	Preferred Operating Region
POS	Period of Simulation
ppb	parts per billion
Project	Reservoir A-1 Project
PSD	Prevention of Significant Deterioration
psi	pounds per square inch
psig	pounds per square inch gauge
psf	pounds per square foot
PV	present value
PVC	polyvinyl chloride
PW	Present worth
PZB	Planning and Zoning Board
RCC	Roller Compacted Concrete
RCRA	Resource Conservation and Recovery Act
RO	Reverse Osmosis
rpm	rotations per minute
RTD	Resistance Temperature Detector
RTU	Remote Terminal Unit
SAD	South Athoutic Division
SAE	Society of Automotive Engineers
SCADA	Supervisory Control and Data Acquisition
STC	Sound Transmission Coefficient
SFWMD	South Florida Water Management District
SFWMM	South Florida Water Management Model
SHPO	State Historic Preservation Office
SPCC	Spill Prevention , Control and Counter Measure
SPDT	Single Pole, Double Throw
SPF	Standard Project Flood

SPT	Standard Penetration Test
SRCO	Site Rehabilitation Completion Order
SSPC	Steel Structures Painting Council
STA	Stormwater Treatment Area
STC	South Transmission Coefficient
SWPPP	Storm Water Pollution Prevention Plan
TC1	Test Cell Number 1
TC2	Test Cell Number 2
TDH	Total Dynamic Head
TFN	Thermoplastic Fixture Wire Nylon Jacketed
THHN	Thermoplastic high heat resistant nylon coated
THWN	Thermoplastic heat and water resistant nylon coated
TW	Tailwater
UL	Underwriters Laboratories IAC
ULD	Unified Land Development Code
UMAM	Uniform Mitigation Assessment Method
US	United States
USACE	United States Army Corp of Engineers
USBR	United States Bureau of Reclamation
USCG	United States Coast Guard
USCOE	United States Army Corp of Engineers
USEPA	Unites States Environmental Protection Agency
USFWS	United States Fish & Wildlife Service
USGS	United States Geological Survey
UST	Underground Storage Tank
Vc	Concrete shear capacity based on ultimate strength design
Vu	Shear in concrete section based on
WBM	Water Balance Model
WCA	Water Conservation Areas
WMA	Wildlife Management Area
WO	Work Order
WRAC	Water Resources Advisory Commission
WRAP	Wetland Rapid Assessment Procedure
WRDA	Water Resource Development Act
WSE	Water Surface Elevation

EXECUTIVE

SUMMARY

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ES-1. EXECUTIVE SUMMARY

ES-1.1 OVERVIEW OF RECOMMENDED PROJECT

The Everglades Agricultural Area (EAA) Reservoir A-1 Project includes the following components:

- Approximately 190,000 acre-feet of storage EAA Reservoir A-1 with a perimeter embankment and seepage canals
- Construction of a northeast pump station [3,600 cubic feet per second (cfs) capacity that pumps from North New River Canal (NNRC)]
- A connector canal from the NNRC to the new northeast pump station
- Evaluation of potential modifications to the existing G-370 pump station, (a 2,775 cfs pump station that currently pumps from the NNRC to the Stormwater Treatment Area 3/4 (STA-3/4) Supply Canal)
- Evaluation of potential modifications to the existing G-372 pump station, (a 3,700 cfs pump station that currently pumps from the Miami Canal to the STA-3/4 Supply Canal)
- Gated discharge structures
- Seepage pump stations
- Two new two-lane bridges on U.S. 27 across the new connector canal
- Improvements to conveyance capacity in the NNRC

The Opinion of Probable Cost for the recommended project, excluding contingency, is $401,000,000 and with cost contingencies is $482,900,000. The costs are summarized in Table ES1.1-1.

The total probable budget depends on the contingency applied to the Opinion of Probable Cost. The Design Criteria Memorandum (DCM) for estimating cost requires a 30 percent contingency for Basis of Design Report (BODR) level costing. However, the major costs in this Project are the embankment and northeast pump station. The embankment was studied extensively and design was based on the Test Cell Project analysis. The northeast pump station will be similar in design to existing pump stations and the Opinion of Probable Cost for it is based on detailed quantity take offs. Therefore, it is recommended that the contingency for these two major cost items be reduced to 20 percent. Therefore, the overall budgeted costs would be $483,000,000.

Table ES-1.1-1 Summary Opinion of Probable Cost

Project Component	Description	Direct Cost (millions)	Indirect Costs (millions)			Total Cost (millions)
			Construction Indirects*	Project Reserve	Contingency	
Embankment and EAA Reservoir A-1	Excavation Embankment Slope Protection Seepage Cutoff Seepage Canal Rock Processing Imported Materials	$ 206.6	$ 89.0	$ 14.8	$ 62.1	**$ 372.5**
Northeast Pump Station	Pump Station Structures Pumps (6) Mechanical Equipment Electrical Equipment Connection Canal Site Work	$ 50.3	$ 16.8	$ 3.3	$ 14.1	**$ 84.5**
Control Structures	Southwest Gate Structure Southeast Gate Structure Northeast Gate Structure and Spillway	$ 10.4	$ 3.5	$ 0.7	$ 4.4	**$ 19.0**
U.S. 27 Bridge	Bridges (2)	$ 3.8	$ 1.2	$ 0.3	$ 1.6	**$ 6.9**
	Totals	**$ 271.1**	**$ 110.5**	**$ 19.1**	**$ 82.2**	**$ 482.9**

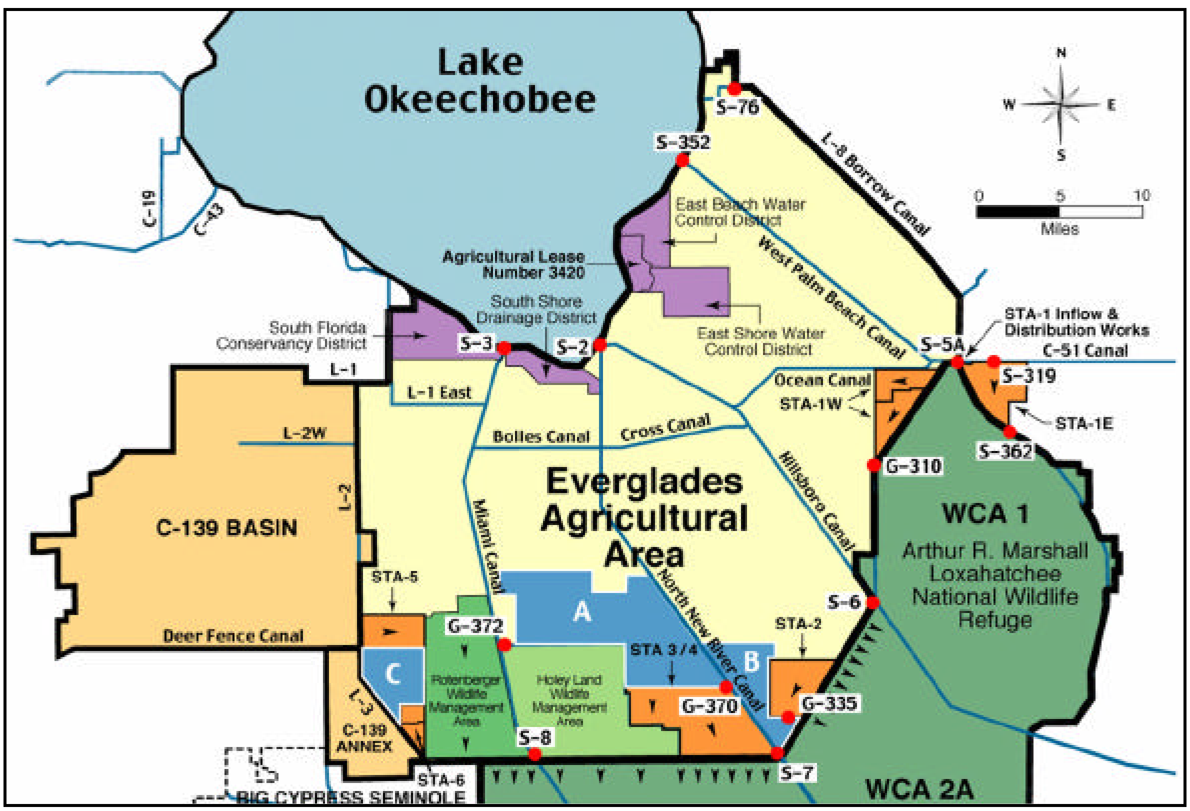
***** Construction indirect costs include sales tax, general requirements, overhead and profit, and bonds and insurance.

ES-1.2 PURPOSE AND OBJECTIVES

The purpose of the EAA Storage Reservoir Project, as defined in the Comprehensive Everglades Restoration Plan (CERP), is to capture EAA basin runoff and regulatory releases from Lake Okeechobee, improve the timing of environmental water supply deliveries to Water Conservation Areas (WCAs) through STA-3/4, meet supplemental agricultural deliveries, reduce Lake Okeechobee regulatory releases to the estuaries, and increase flood protection within the EAA.

In October 2003, the South Florida Water Management District (SFWMD) decided to pursue a "Dual Track" for the EAA Storage Reservoir Project. While the multi-agency Project Delivery Team, led by the US Army Corps of Engineers (USACE), continues to develop the Draft Integrated Project Implementation Report (PIR), the SFWMD is proceeding with the design and construction of a reservoir located in the Everglades Agricultural Area on the land known as the Talisman Exchange. The PIR considers compartment A of the EAA Reservoir Storage Project, which will contain 360,000 acre-feet of water. The SFWMD's current focus is on the first phase of compartment A, or EAA Reservoir A-1, which will store 190,000 acre-feet of water. The regional project overview for the EAA Reservoir A-1 is shown on Figure ES-1.2-1.

Figure ES-1.2-1 Regional Project Overview



Implementation of the EAA Reservoir A-1 Project will meet objectives consistent with the ongoing work by the USACE. In accordance with the USACE's Draft PIR, the objectives of the full reservoir project include:

- Reduction of the Lake Okeechobee regulatory releases to the estuaries and reduce backpumping runoff from the Study Area into Lake Okeechobee by sending the water to the EAA Reservoir A-1
- Improved environmental releases through the storage of water for later release to the Everglades when needed
- Improved regional water supply for the agricultural community currently served by the EAA canals and other areas served by Lake Okeechobee
- Flow equalization and optimization of treatment performance of STAs by storing peak storm event discharges in the EAA Reservoir A-1 for controlled release to the Everglades through STA-3/4
- Reduction of flood impacts

The Project benefits (success indicators to meeting the goals and objectives of CERP and plan formulation) are included in the USACE PIR for the EAA Storage Reservoir project. The PIR was published in September, 2005.

The success of the EAA Reservoir A-1 will be judged on how well the Project meets its design objectives where applicable to Compartment A in the PIR. In terms of consistency with the goals and objectives identified in the PIR, the EAA Reservoir A-1 will:

- Provide significant improvement in the water deliveries through the WCAs to STA-3/4
- Provide water for agricultural deliveries not previously available from the system
- Reduce the releases to the estuaries from Lake Okeechobee
- Improve flood protection in the EAA

The EAA Reservoir A-1 will store up to 190,000 acre-feet of water from stormwater runoff and releases from Lake Okeechobee at any given time. Without this Project, that water will potentially cause flooding in the EAA, will need to be pumped to STA-3/4, will be bypassed to tide, or will potentially be released to the estuaries from Lake Okeechobee. Thus there will be a reduction in the potential for flooding and releases to the estuaries once the Project is completed.

Highlighted in the Executive Summary are some of the major considerations that support the recommendations included in this BODR. These include EAA Reservoir A-1 configuration, embankment cross-section, water balance, seepage, operations, pump station sizing, control structures, and probable cost.

ES-1.3 REGULATORY CONSIDERATIONS AND PROJECT ASSURANCES

Within the Water Resources Development Act of 2000 (WRDA 2000), the U.S. Congress approved CERP's objectives to restore, preserve, and protect the south Florida ecosystem while providing for water-related needs of the region. The components of CERP will increase storage and water supply for the natural system, as well as for agricultural and urban needs. Implementation of CERP must also be consistent with State law. As a local sponsor, SFWMD has responsibilities that are outlined in Section 373.1501(5) of the Florida Statutes (F.S.).

After the EAA Reservoir A-1 Project preferred alternative is selected by the SFWMD and the spatial extent of the EAA Reservoir A-1 effects is identified, separate comparisons of modeling simulations will be performed to satisfy the federal (WRDA 2000) and State (Section 373.1501 F.S.) assurances, and to identify the water available for the protection of fish and wildlife and for other water related needs. Separate comparative analyses are planned by the SFWMD to evaluate the following conditions:

- Section 373.1501 F.S. - Assurances analysis to evaluate the quantity of water available to existing legal users
- Section 373.1501 F.S. - Assurances analysis to evaluate the effects of the EAA Reservoir A-1 Project implementation on existing levels of flood protection
- WRDA 2000 - Quantification of water made available by the EAA Reservoir A-1 Project for the protection of fish and wildlife
- WRDA 2000 - Quantification of water made available by the EAA Reservoir A-1 Project for other water related needs

Project assurance had been considered in a preliminary manner as part of the evaluation of engineering alternatives for the EAA Reservoir A-1 in the BODR. The assurances as addressed in sections of the BODR are as follows:

- Alternatives to control seepage from the EAA Reservoir A-1 and provide protection from flooding due to seepage are described in Section 9.
- Modeling results, which describe the environmental deliveries to the WCAs, are presented in Section 6.
- Modeling results, which describe the agricultural deliveries to the farmlands are presented in Section 6.
- The EAA Reservoir A-1 will be operated to store water, which would otherwise be lost to tide or sent to the WCAs during wet seasons. This is presented in the Operations Plan described in Section 20.

ES-1.4 WATER BALANCE

A water balance analysis was performed for the EAA Reservoir A-1 to assess the hydrologic and hydraulic components of the system. The analysis was performed with a water balance model (WBM) developed by Black & Veatch Corporation (Black & Veatch), which was developed to analyze the EAA Reservoir A-1's storage capacity and operations on a daily basis. A period of simulation (POS) was used, which employed meteorological records for 36 years from January 1, 1965 to December 31, 2000. The WBM was based on the South Florida Water Management Model (SFWMM), Everglades Construction Project (ECP) 2015 and 2010 simulation (version 5.4.2).

The WBM was used to optimize the storage capacity of the EAA Reservoir A-1, while evaluating the impacts to flows in the NNRC, Miami Canal, and the STA-3/4 Supply Canal. In addition, the model was used to evaluate pumping facility locations and sizing, and the distribution of releases from the EAA Reservoir A-1 for environmental and agricultural and sizing purposes.

For the recommended reservoir configuration, based on the ECP 2015 run, the average annual environmental delivery supplied by the EAA Reservoir A-1 via STA-3/4 is approximately 685,000 acre-feet over the POS, with a maximum of 1,486,746 acre-feet in water year 1983 and a minimum of 103,685 acre-feet in water year 1990. The current average annual inflow to STA-3/4 is approximately 656,000 acre-feet (Piccone, 2005). The total deliveries over the POS are approximately 24,000,000 acre-feet.

For the recommended reservoir configuration, based on the ECP 2010 run, the average annual agricultural delivery supplied by the EAA Reservoir A-1 is 84,000 acre-feet over the POS, with a maximum of 159,764 acre-feet in water year 1985 and a minimum of 18,922 acre-feet in water year 1970. The total deliveries are approximately 3,000,000 acre-feet.

Figure ES-1.4-1 illustrates the average annual inflows and outflows of the EAA Reservoir A-1. In addition, Figure ES-1.4-2, Figure ES-1.4-3 and Figure ES-1.4-4 illustrate the EAA Reservoir A-1 operations for the selected "Average" (1991-1992), "Wet" (1977 and 1978), and "Dry" period of water years (1971 and 1972), respectively. The performance of the EAA Reservoir A-1 for selected water years is also provided in Table 1.4-1.

Figure ES-1.4-1 Average Annual Inflows and Outflows of the EAA Reservoir A-1

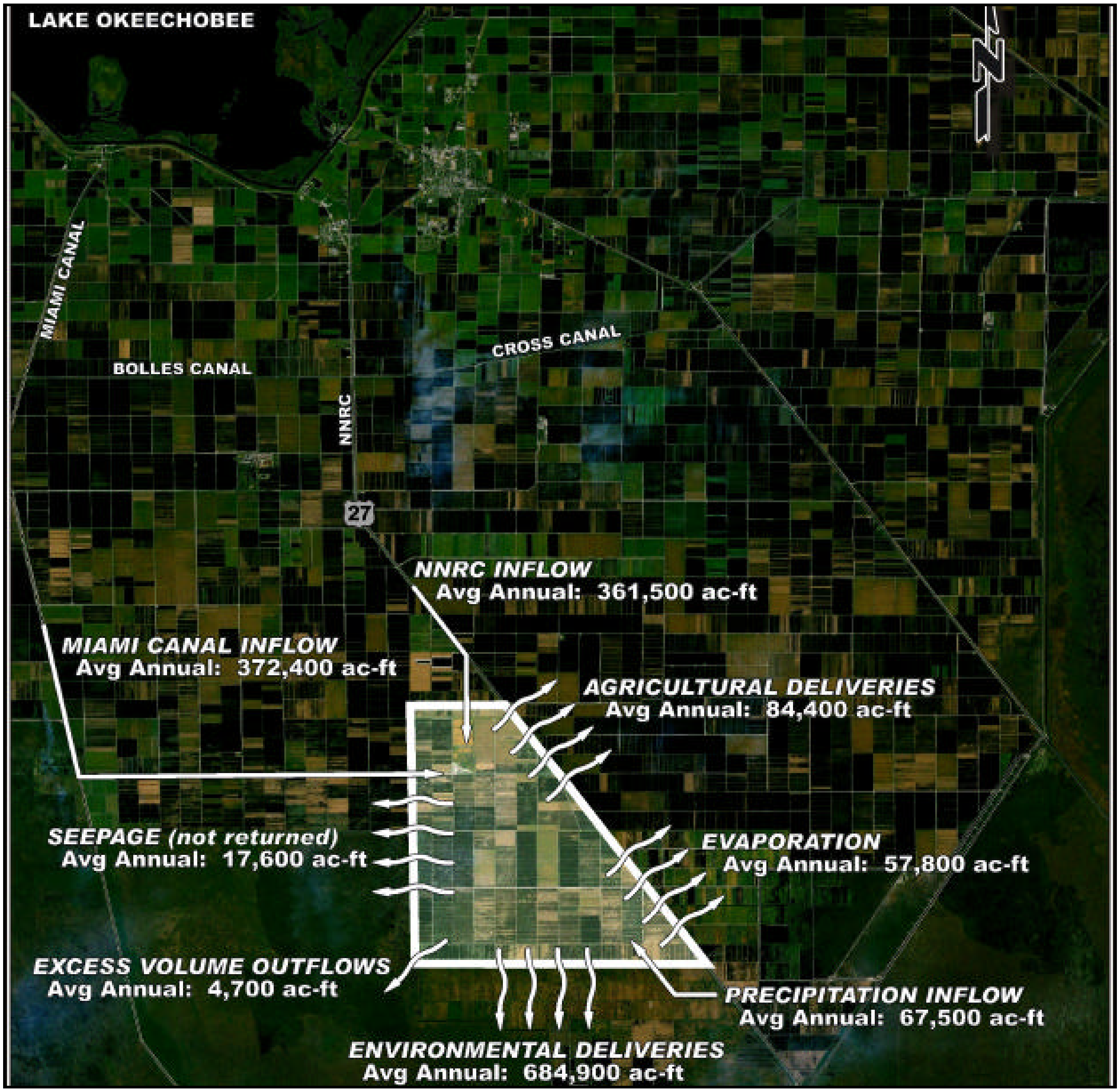


Figure ES-1.4-2 EAA Reservoir A-1 Operation During an "Average" Water Year

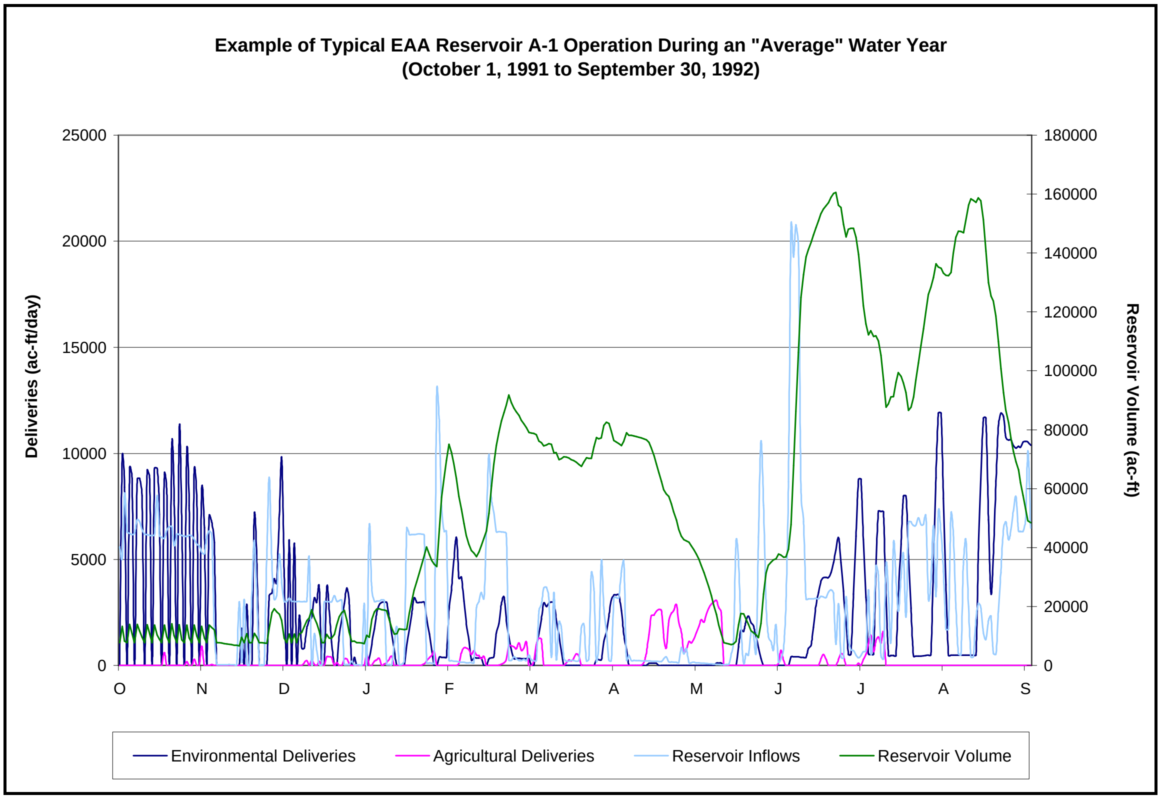


Figure ES-1.4-3 EAA Reservoir A-1 Operation During a "Wet" Water Year

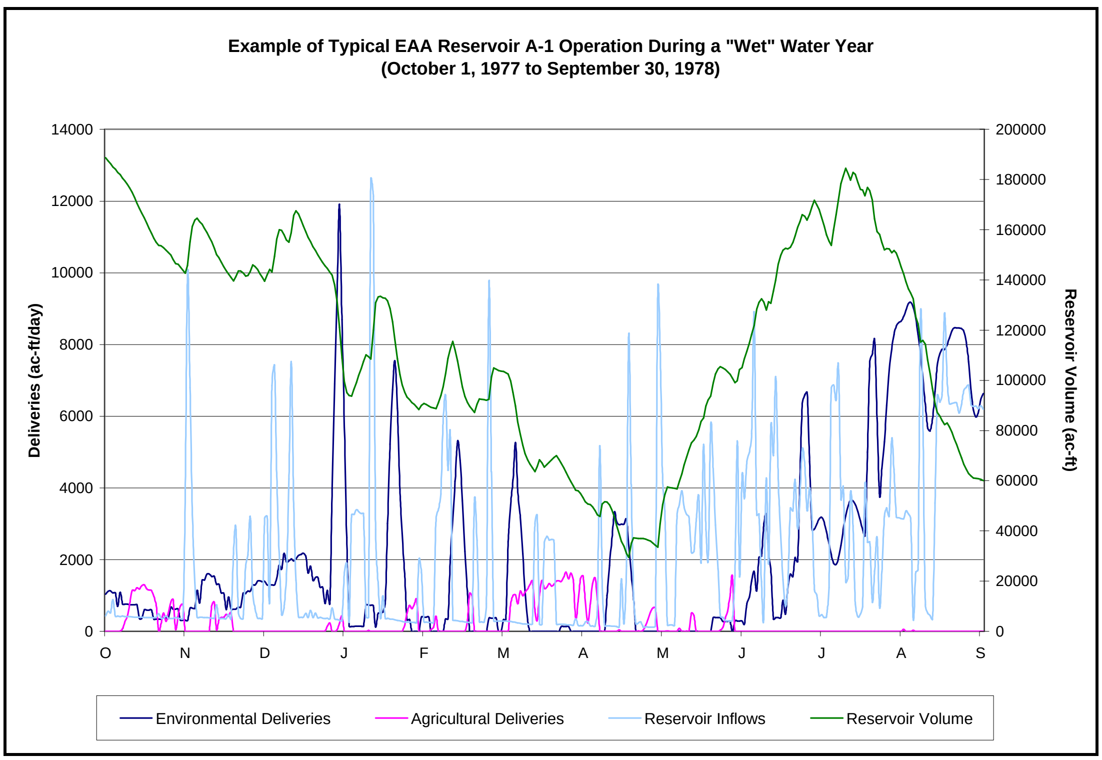


Figure ES-1.4-4 EAA Reservoir A-1 Operation During a "Dry" Water Year

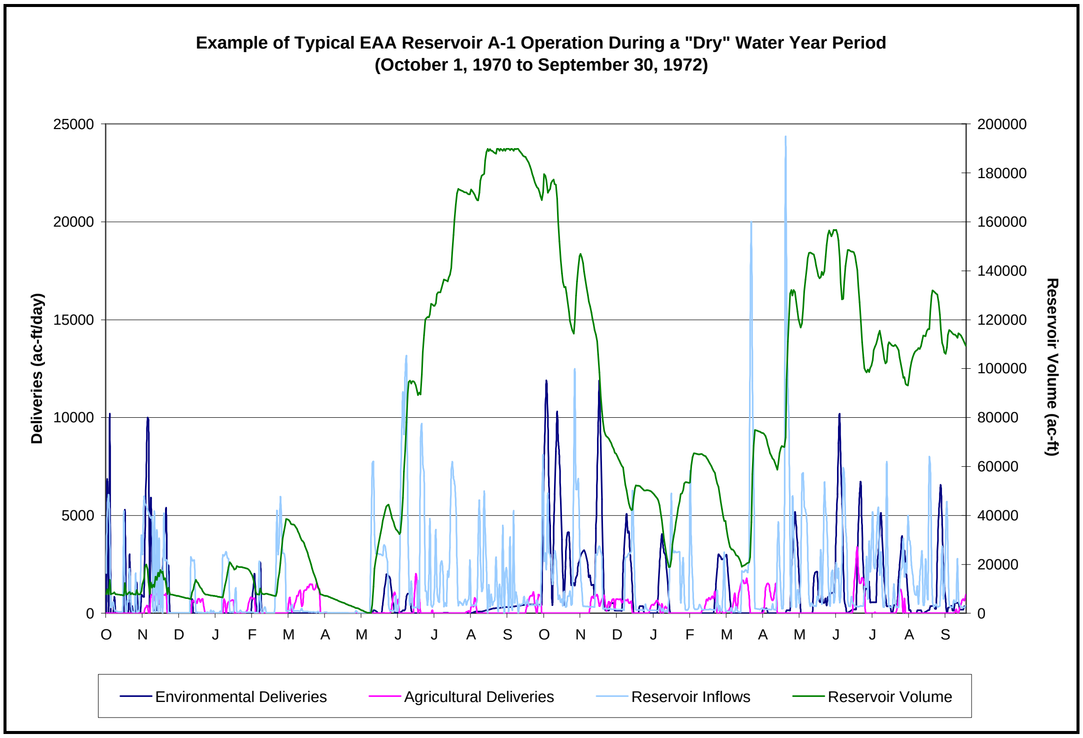


Table 1.4-1 Summary of Performance of the EAA Reservoir A-1 for Selected Water Years

		1971 "Dry" Year	1992 "Average" Year	1978 "Wet" Year	Complete POS
EAA Reservoir A-1 Inflows	NNRC inflow, acre-feet	179,165	470,155	346,118	12,906,675
	Miami Canal inflow, acre-feet	227,050	529,297	233,841	13,229,975
	Precipitation, acre-feet	53,443	71,879	78,144	2,423,429
EAA Reservoir A-1 Outflows	Environmental deliveries, acre-feet	123,023	857,780	624,402	22,518,200
	Agricultural deliveries, acre-feet	74,451	103,491	80,440	3,073,453
	Evaporation, acre-feet	59,730	62,708	52,156	2,081,752
	Seepage, acre-feet	22,687	10,913	30,567	639,218
	Excess volume outflows, acre-feet	12,599	0	0	185,494
EAA Reservoir A-1 Volumes	Start of year (BOD), acre-feet	7,827	7,827	188,775	0
	End of year (EOD), acre-feet	174,995	44,266	59,313	62,451
Environmental and Agricultural Deliveries	Environmental deliveries, acre-feet	360,492	1,243,166	772,530	31,778,063
	Environmental deliveries supplied by canals, acre-feet	9,438	71,506	148,079	1,694,324
	Percentage of environmental deliveries met	37%	75%	100%	76%
	Agricultural deliveries, acre-feet	176,933	126,313	80,440	4,755,705
	Percentage of agricultural deliveries met	42%	82%	100%	65%

ES-1.5 SEEPAGE CONTROL

As with other surface water features such as STA and canals, seepage will occur from EAA Reservoir A-1 because the soil is permeable to approximately 200 feet below the surface of the site. Both three-dimensional MODFLOW groundwater modeling and two-dimensional SEEP/W groundwater modeling were performed to analyze seepage from EAA Reservoir A-1. The groundwater models were used to evaluate the following major issues:

- The effect of seepage on embankment stability
- The amount of water the EAA Reservoir A-1 loses to seepage
- The percentage of seepage that can be collected and returned to the EAA Reservoir A-1
- The effectiveness of various seepage control alternatives
- The amount of unrecoverable seepage, if any, that migrates to surrounding areas from various seepage control alternatives
- The effect of any unrecoverable seepage on groundwater levels in the surrounding areas.

Although quite effective at reducing seepage, cutoff walls of practical depth cannot completely eliminate seepage from EAA Reservoir A-1. Additional seepage control measures were considered, including the effect of lowering the water level in the seepage canal as a way to draw seepage to the surface and the use of pressure-relief wells to intercept deep seepage before it migrates to surrounding areas. Five seepage control alternatives were evaluated with MODFLOW.

- Alternative 1 - 34-foot cutoff wall and 13.5-foot deep seepage canal around entire EAA Reservoir A-1, seepage canal water level held 3.5 feet below the groundwater level in adjacent farmland
- Alternative 2 - 34-foot cutoff wall and 10-foot deep seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal water level held at the same level as the groundwater in the adjacent farmland
- Alternative 3 - 34-foot cutoff wall and 13.5-foot deep seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal water level held 3.5 feet below the groundwater level in adjacent farmland
- Alternative 4a - Pressure-relief wells spaced at 100 feet linked together in sets with a total of 21 pump stations of 3,900 gallons per minute (gpm) each; 34-foot cutoff wall and 10-foot deep seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal water level held at the same level as the water level in the adjacent farmland. Alternative 4b includes separate pumps in each well, each with a capacity of approximately 150 gpm, which discharge to the seepage canal.

- Alternative 5a - Pressure-relief wells spaced at 200 feet linked together in sets with a total of 23 pump stations of 3,300 gpm each; 34-foot cutoff wall and 10-foot deep seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal water level held at the same level as the water level in the adjacent farmland. Alternative 5b is similar to Alternative 5a except that it includes separate pumps in each well, each with a capacity of approximately 275 gpm, which discharge to the seepage canal.

Alternative 1, including a 34-foot cutoff wall and a 13.5-foot deep seepage canal surrounding the entire EAA Reservoir A-1 and maintaining the water level of the seepage canal below the level in the surrounding farmlands, would be the most effective of the five alternatives evaluated. However, this alternative includes a significantly higher present worth cost of between $134 to $181 million more than the other alternatives evaluated.

The other seepage control alternatives allow migration of seepage to the Holey Land and STA-3/4, but essentially eliminate impacts to farms and U.S. 27.

Alternative 2 allowed seepage into the farmland and included an estimated cost for the seepage to be pumped off the land by the farmers. Modeling results for Alternative 3 indicate that maintaining the water level of the seepage canal below the water levels in the farmlands effectively prevents offsite migration of seepage. The installation of pressure-relief wells as described by Alternatives 4 and 5 is predicated upon capturing deep seepage at the point where water passes beneath the bottom of the cutoff wall.

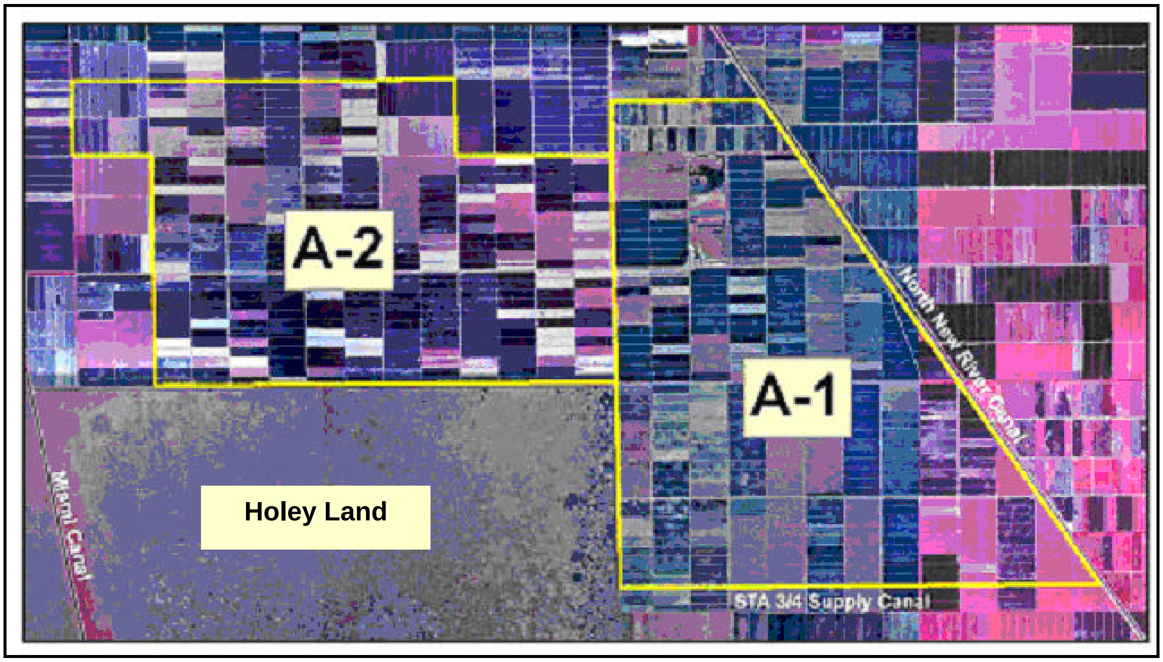
Alternatives 2 and 3 are the lowest cost alternatives. Alternative 3 allows the SFWMD more control of the pumping rates in the seepage canal than alternative 2, which relies on the farmers to pump the seepage. These two alternatives will be further assessed during the EAA Reservoir A-1 design.

A monitoring program of groundwater levels in the farmland should be initiated during the construction of the EAA Reservoir A-1 and continued after construction is completed. This will provide information to the SFWMD for evaluating the effectiveness of the selected seepage control measures. The monitoring program is a means to document whether flood protection (from seepage) has been provided as required by the Project assurances, which are discussed in Section 4.

ES-1.6 RESERVOIR CONFIGURATION

The configuration of the EAA Reservoir A-1 embankment and seepage canals directly affect the total amount of storage for the EAA Reservoir A-1. In order to achieve the storage requirement of 190,000 acre-feet, setback requirements were balanced with the total area available to meet this requirement. This is presented in greater detail with respect to SFWMD, USACE, and U.S. Fish & Wildlife Service (USFWS) input, construction considerations, cost, and other factors in Section 8. Setbacks for each side of the EAA Reservoir A-1 embankment are summarized below. The configuration provides storage in EAA Reservoir A-1 of 190,000 acre-feet at a depth of 12 feet and an EAA Reservoir A-1 footprint area of approximately 16,000 acres. The limits of the land for the proposed EAA Reservoir A-1 and EAA Reservoir A-2 site are shown on Figure ES-1.6-1. The configuration provides storage in EAA Reservoirs A-1 and A-2 of 360,000 acre-feet.

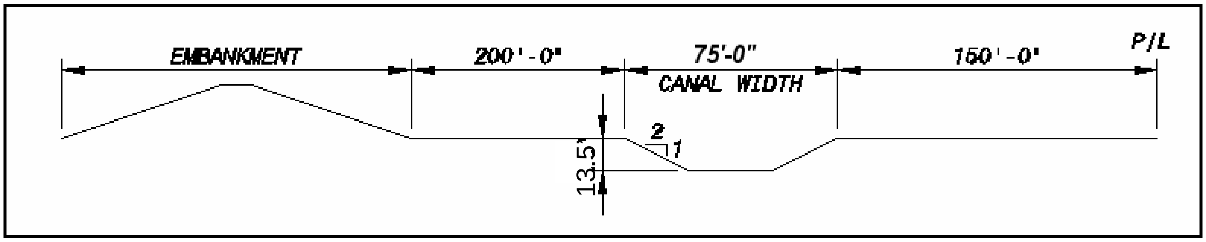
Figure ES-1.6-1 Reservoir Parcels A-1 and A-2



North Boundary and North Portion of West Boundary (Portion North of Future Reservoir A-2)

- 150-foot setback from EAA Reservoir A-1 boundary to the seepage canal
- 75-feet wide seepage canal
- 200-foot setback from seepage canal to the outside toe of the embankment for construction stockpiling and future wetland areas
- 300-foot setback from the inside toe of the embankment to the internal borrow excavation
- The cross-section for these setbacks is shown on Figure ES-1.6-2

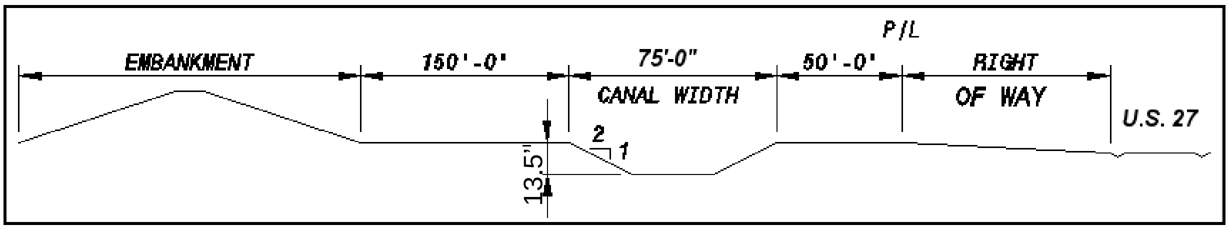
Figure ES-1.6-2 North Boundary Setbacks



East Boundary (Portion Adjacent to U.S. 27)

- 50-foot setback from highway right-of-way to the seepage canal
- 75-foot wide seepage canal
- 150-foot setback from seepage canal to the outside toe of the embankment for construction stockpiling and future wetland areas
- 300-foot setback from the inside toe of the embankment to the internal borrow excavation
- The cross-section for these setbacks is shown on Figure ES-1.6-3.

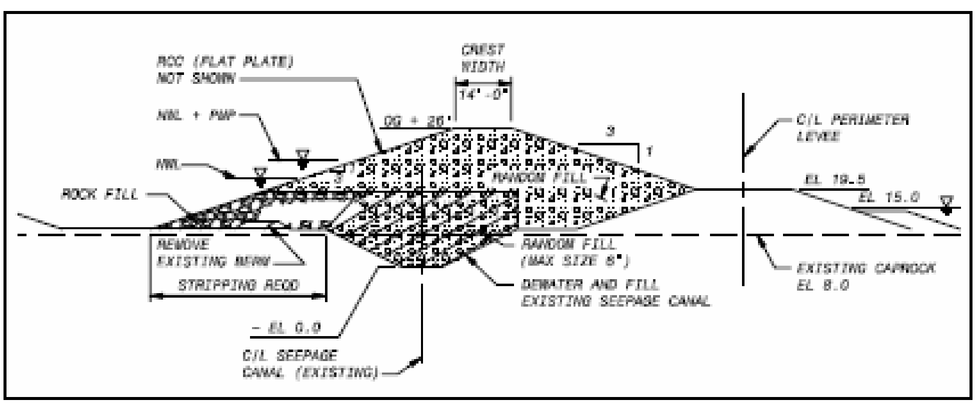
Figure ES-1.6-3 East Boundary Setbacks



South Boundary and South Portion of West Boundary (Portion Adjacent to the STA-3/4 Supply Canal)

To minimize cost and maximize storage, the configuration along the STA-3/4 Supply Canal will incorporate the northern embankment of the Supply Canal. This configuration is discussed in further detail in Section 8 of this BODR. A 300-foot setback from the inside toe of the embankment to the internal borrow excavation will apply. The cross-section for this embankment is shown on Figure ES-1.6-4; for clarity, the drawing does not show the extent of the setback.

Figure ES-1.6-4 Zoned Embankment along STA-3/4 Supply Canal

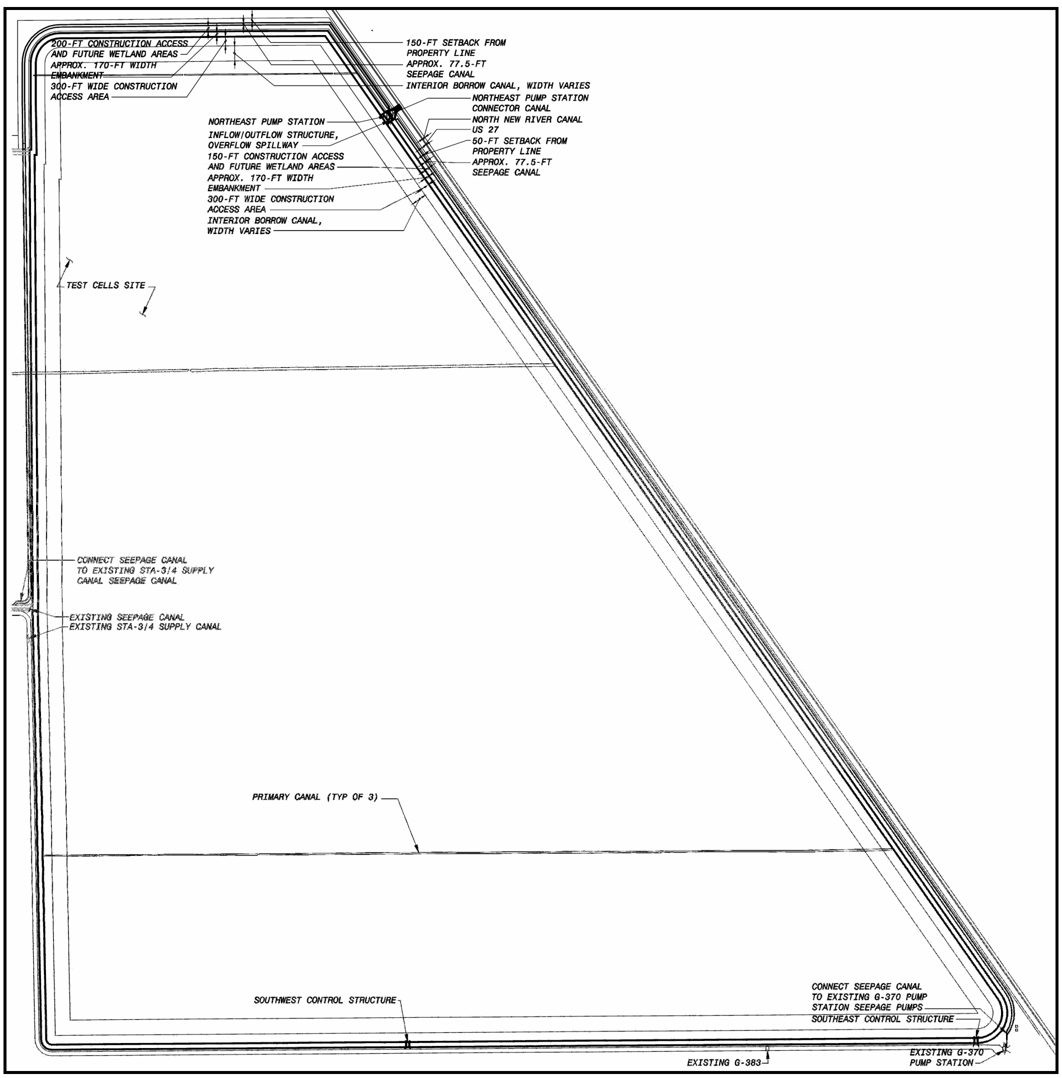


Boundary Adjacent to Future Reservoir A-2

- 75 feet wide seepage canal
- 150-feet wide setback from seepage canal to the outside toe of the embankment for construction stockpiling
- 300 feet setback from the inside toe of the embankment to the internal Borrow Canal

Curved corners provide an additional benefit and will be utilized in the northwest and southeast corners of EAA Reservoir A-1. Both the northwest and southeast corners will be curved at a radius that aids construction of the embankment. Because of the acute angle, in the southeast corner, an embankment configuration that parallels the EAA Reservoir A-1 property line adds little additional storage. Therefore, attention will be given to cost when selecting the radius in the southeast. Additionally, the radius of the southeast corner must be sufficient to prevent relocation of existing facilities, including existing helipads. An overall site layout is shown in Figure ES-1.6-5

Figure ES-1.6-5 Overall Site Layout



ES-1.7 EMBANKMENT CROSS-SECTION

Two fundamental types of embankments were considered for this site:

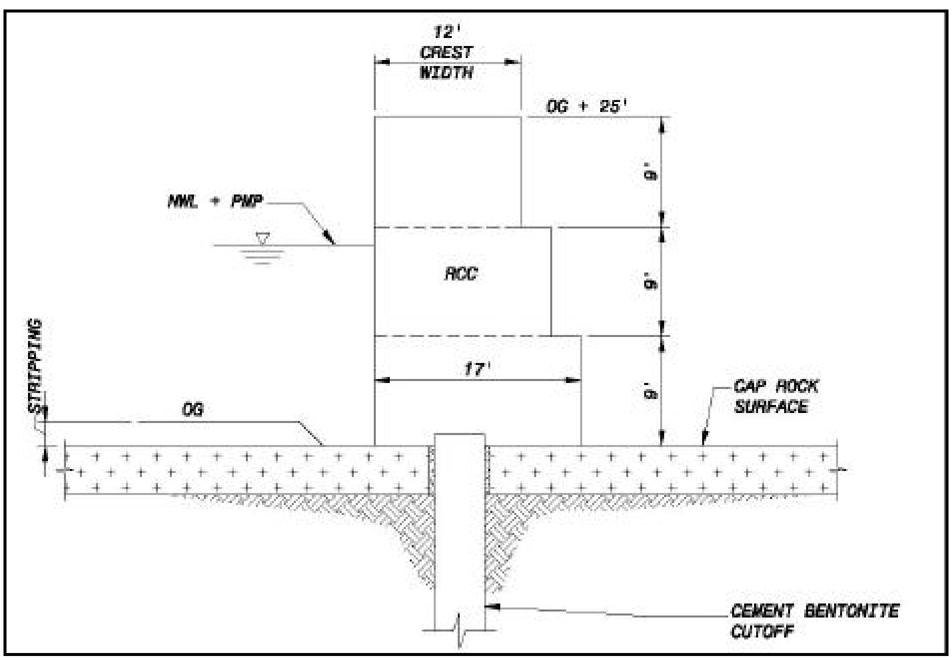
- A concrete gravity type dam using roller compacted concrete (RCC)
- A zoned embankment dam

Each type was considered in detail during the preparation of this BODR. A number of alternative arrangements for each type were considered and an opinion of probable cost prepared to evaluate the cost effective aspects of each alternative. The advantages, disadvantages, and risks of each section were considered. A summary of the evaluations and findings are presented in Section 8 of the BODR.

The evaluation of alternatives must consider initial and long-term stability, seepage control, foundation conditions, and probable costs with appropriate allowances for risks, uncertainties and the cost of mitigation measures. The construction sequence and requirements for each alternative has been considered in detail. The most favorable concrete dam and embankment sections are presented below.

The roller compacted concrete gravity dam section depicted in the most recent Tentatively Selected Plan prepared by USACE is composed of a three stepped RCC section with a vertical face on the interior of the dam. The advantages and disadvantages of the RCC dam are presented in Section 8. The cross-section for the RCC dam is shown in Figure ES-1.7-1.

Figure ES-1.7-1 Cross-Section of Roller Compacted Concrete Dam



A zoned embankment concept has been developed to utilize materials from the required seepage collection canal excavations and available on-site borrow resources, and to minimize sorting and processing of the excavated materials for embankment construction. The rockfill zone material will be produced from the caprock providing structural stability to the upstream slope. The advantages and disadvantages of a zoned embankment are presented in Section 8. The cross-section for the zoned embankment is shown in Figure ES-1.7-2, and the cross section for the zoned embankment along STA-3/4 is shown in Figure ES-1.7-3.

Figure ES-1.7-2 Zoned Embankment Cross-Section

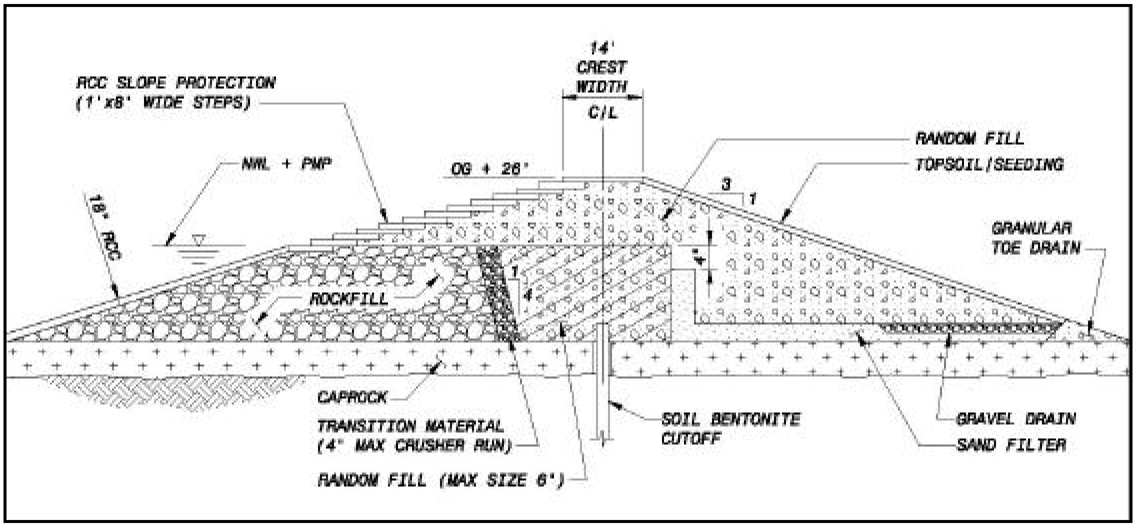
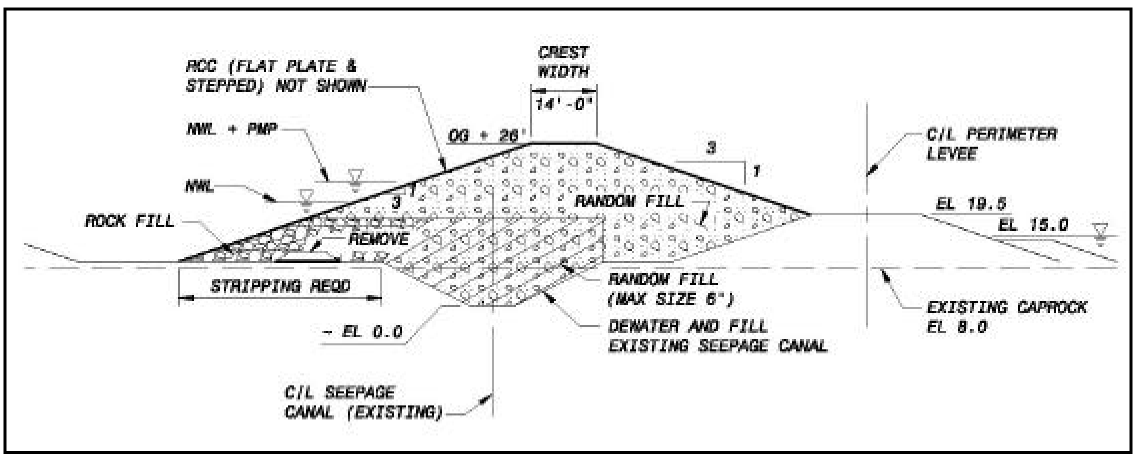


Figure ES-1.7-3 Zoned Embankment Cross-section along STA-3/4 Supply Canal



The purpose of the foundation seepage control is to mitigate seepage losses from the EAA Reservoir A-1, protect the foundation from possible damage by piping, and minimize excess uplift pressures to enhance stability. With higher head in the EAA Reservoir A-1, foundation stability issues are more critical and economic impacts due to pumping would experience on a long-term basis. Several different configurations to mitigate seepage from the EAA Reservoir A-1 and control exit pressures were evaluated: adequately sized key trench, cutoff wall or upstream blanket, and increasing the distance between the EAA Reservoir A-1 and seepage collection canal. In view of the potential for piping, a foundation cutoff wall to a minimum depth of 34 feet, or the base of the Fort Thompson Formation, is recommended for seepage control.

Based on the results of the technical and cost evaluation, the least cost alternative shown in Figure ES-1.7-2 and Figure ES-1.7-3 are preferred alternative to be advanced to 30 percent design.

ES-1.8 PUMP STATION AND DISCHARGE STRUCTURES

ES-1.8.1 Pump Station

Figure ES-1.8-1 and Figure ES-1.8-2 show potential layouts for gates and pump stations at the EAA Reservoir A-1. Seven pumping and discharge alternatives were selected for preliminary consideration. In general, all alternatives except one were based on the addition of a new northeast pump station located adjacent to the NNRC in the northeast corner of the EAA Reservoir A-1 site.

Figure ES-1.8-1 Pumping and Discharge Facilities - Alternative No. 2

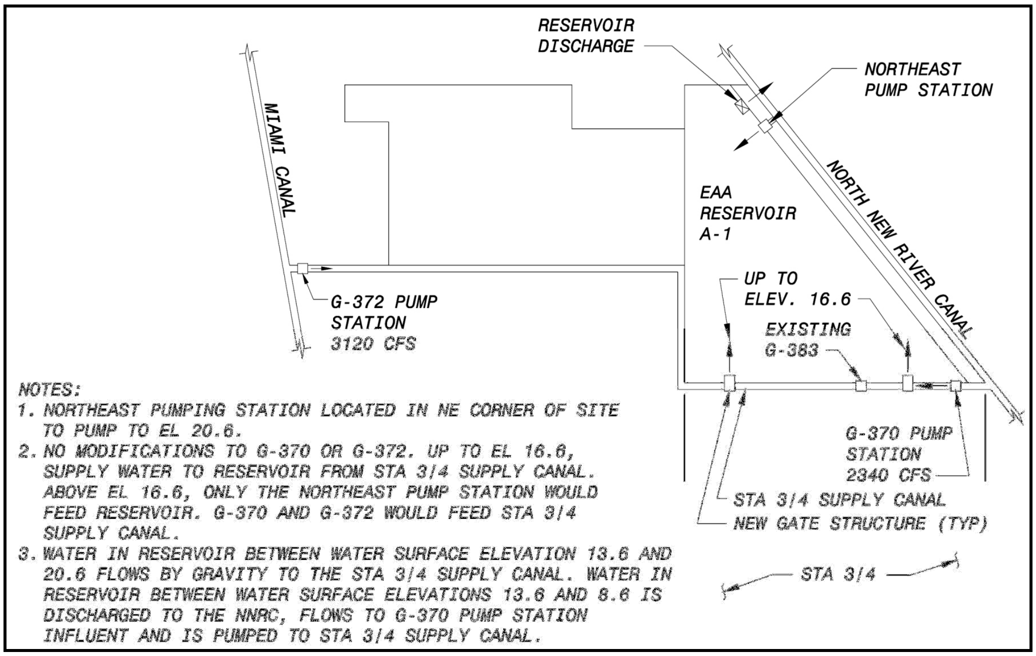
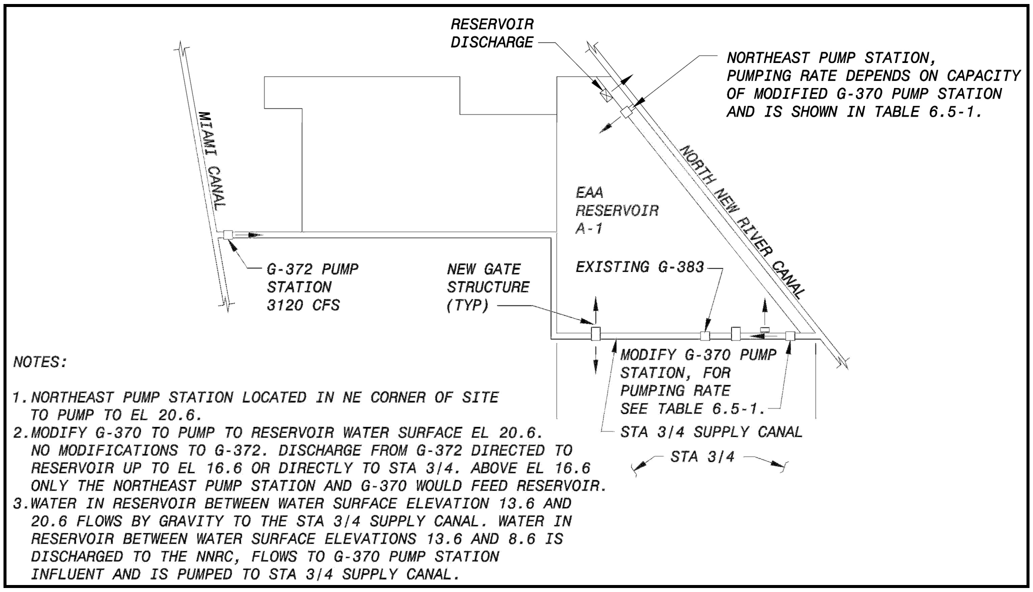


Figure ES-1.8-2 Pumping and Discharge Facilities - Alternative No. 3



The operating level of the EAA Reservoir A-1 will fluctuate between elevation 8.6 and 20.6 NAVD88. The normal and maximum design operating elevations of the STA-3/4 Supply Canal are 13.6 and 16.6 NAVD88, respectively. Both G-370 and G-372 pump stations are designed to pump to these elevations. However, pumping to elevation 16.6 NAVD88 will rapidly diminish their respective design capacities. While it is possible to partially supply the EAA Reservoir A-1 from G-370 and G-372 pump stations without modifications, significant modifications would be required to pump to the full EAA Reservoir A-1 elevation of 20.6 NAVD88.

Further evaluations were completed using the WBM in an effort to optimize the performance of the pump stations and EAA Reservoir A-1. Two fundamental optimization criteria were considered:

- Optimization based on effectiveness of supplying environmental and agricultural deliveries
- Optimization based on effectiveness of capturing priority water sources. Listed in order of descending priority the water sources are runoff from the NNRC drainage and backpumping to Lake Okeechobee, and Lake Okeechobee releases.

Optimization was considered to be achieved when further increases in the size of the northeast pump station no longer provided significant benefit. Numerous combinations of northeast pump station sizes and modifications to existing G-370 and G-372 pump stations were evaluated.

Evaluations found that the optimization goal of installing the most cost effective pump capacity to meet maximum deliveries ran counter to the goal of maximizing pump capacity to capture local runoff, pump backs, and Lake Okeechobee regulatory releases:

- A northeast pump station sized for 1,500 cfs working with G-370 and G-372 pump stations unmodified would provide sufficient capacity during the first phase of operation to provide the maximum delivery percentages that can be expected with an EAA Reservoir A-1 of 190,000 acre-feet of storage volume. Further modifications to the NNRC and to G-370 and G-372 pump stations to allow pumping capacities of 2,220 and 3,700 cfs respectively to full EAA Reservoir A-1 water depth would provide the additional capacity needed for the second phase of operation. A conveyance capacity of up to 6,400 cfs would be required in the NNRC depending on the alternative selected.

- A northeast pump station sized for 3,500 to 4,000 cfs working with G-370 and G-372 pump stations unmodified would be required to provide sufficient capacity during the first phase to maximize local runoff and pump back capture. NNRC modifications would also be required to increase conveyance capacity. Further modifications to G-370 and G-372 pump stations to allow pumping capacities to full EAA Reservoir A-1 water depth would provide the additional capacity needed for the second phase of operation.

The second alternative has several advantages over the first:

- The larger pump station can meet all of the delivery goals that a 1,500 cfs pump station would meet but the smaller station could not provide the same priority removal levels.
- A 3,500-4,000 cfs pump station would provide a significant increase in flood protection capacity.
- Having a substantial pumping capability in the northeast pump station will ease the disruption that will be experienced during the phase two modifications of G-370 and G-372 pump stations.
- The larger pump station allows for capture of most storm related peak flows.

The primary disadvantage for the larger pump station alternative is cost (about $15.4 million). In addition to the costs associated with a smaller pump station, the 1,500 cfs pump station can provide the optimum deliveries without canal modification for the first phase, and minimal canal modification for the second phase. To be effective, the larger pump station would require canal modifications that coincide with the first phase of construction.

Northeast pump station recommendation:

- Construction of a 3,600 cfs northeast pump station concurrent to the construction of EAA Reservoir A-1.
- Use of the G-370 and G-372 pump stations unmodified during phase one operation to pump into the EAA Reservoir A-1 when its water levels are less than eight feet and directly to the STA-3/4 Supply Canal when EAA Reservoir A-1 water levels are greater than eight feet.
- Modification of the G-370 and G-372 pump stations to pump 2,220 and 3,700 cfs, respectively, to full EAA Reservoir A-1 depth as part of the construction of phase two (EAA Reservoir A-2).
- Canal modifications to provide matching conveyance capacity, with an associated cost of up to $37 million depending on the alternative selected.

ES-1.9 DISCHARGE STRUCTURES

Three gate structures are required for implementation of the recommended alternative as shown on Figures ES-13 and ES-14. Two structures located in the southern embankment would be required to provide a dual function of EAA Reservoir A-1 filling and environmental deliveries from the EAA Reservoir A-1. Some studies have suggested that a water quality benefit may result from passing water through EAA Reservoir A-1 prior to discharging to STA-3/4. Additional structures would be required in order to achieve the potential water quality benefit. Implementation of the latter alternative would increase the project cost by approximately $26 million. Because of the added cost and a relatively small (13 to 17 percent reduction in phosphorus loading) water quality benefit, the three gate structure option is recommended. Even with the recommended alternative, operational strategies can be implemented to achieve the goal of routing much of the water through the EAA Reservoir A-1 before passing to the STA-3/4.

In addition to the gate structures the EAA Reservoir A-1 will be equipped with an orifice-type spillway which will guard against overfilling. The spillway will be designed to limit overflow discharges to less than 500 cfs during rainfall events with less than a 100-year recurrence interval.

ES-1.10 OPERATIONS

A Project Operating Manual (POM), for day-to-day use in managing essentially all foreseeable conditions affecting the EAA Reservoir A-1 will be developed upon completion of the Project. However, a draft has been prepared as part of this BODR for use during the Draft Integrated Project Implementation Report/Environmental Impact Statement (PIR/EIS) phase of the EAA Reservoir A-1 Project. The manual will be modified and revised, as necessary, through several Project phases. During the detailed design phase, it will be modified to define any temporary operations to be used during construction, including start-up and filling. The Operation Manual for STA-3/4 will also be modified as required, to reflect operations during periods when construction in the embankments for the Inflow and Supply Canals could disrupt operations.

Knowledge gained from the Operational Testing and Monitoring Phase will then be incorporated into the Draft POM, which will be coordinated with SFWMD and the USACE Jacksonville District (SAJ), and will supersede all other iterations of the Draft POM. The final version of the POM will be used by SFWMD when they accept responsibility for long term operations of the EAA Reservoir A-1.

The EAA Reservoir A-1 will store runoff that would otherwise have gone to tide and will improve the quantity, timing, and distribution of water deliveries to the environment. It has been demonstrated using an area specific computer model, applied to a period of simulation data from the SFWMM (which is the same as the ECP 2010 and 2015 version 5.4.2 model), that approximately 685,000 acre-feet per year of water can be delivered to the environment by EAA Reservoir A-1. Operating criteria for EAA Reservoir A-1 will be developed in subsequent versions of this POM to be consistent with the water reservations or allocations for the natural system made by the State in accordance with the WRDA 2000.

EAA Reservoir A-1 will also provide water for substantial agricultural deliveries by capturing storm runoff. Agricultural deliveries that cannot be met by the EAA Reservoir A-1 will continue to be supplied from Lake Okeechobee. When water is available in the EAA Reservoir A-1 for agriculture, it will normally be released through the northeast gate structure located near the northeast pump station from where it will flow to the NNRC via the connector canal from the pump station. When the EAA Reservoir A-1 water level is below that needed for gravity flow to the NNRC, pumps located in the northeast pump station will be activated to deliver water to the NNRC. It has been demonstrated using an area specific computer model that a high percentage of the agricultural needs along the NNRC can be provided by EAA Reservoir A-1.

The EAA Reservoir A-1 will be operated to assure that implementation of the project will not diminish flood protection in the EAA. During periods when the EAA Reservoir A-1 contains water and it is necessary to prevent seepage from impacting adjacent properties, the seepage canal water level will be pumped down as required to prevent the groundwater level from rising. A groundwater model has been utilized to verify that lowering the water level in the seepage canal will be effective in preventing flooding of adjacent properties.

The EAA Reservoir A-1 Project will also provide capacity to store storm runoff and will increase the pumping capacity from the NNRC. In addition, areas within the EAA Reservoir A-1 previously used for agriculture will no longer deliver runoff to the NNRC, thereby making available 500 cfs of NNRC water that was previously unavailable. Therefore, the Project will not diminish flood protection and should reduce flooding in the NNRC under most conditions.

SECTION 1

INTRODUCTION

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FIGURES

1. INTRODUCTION

1.1 THE COMPREHENSIVE EVERGLADES RESTORATION PLAN (CERP)

1.1.1 General

The Comprehensive Everglades Restoration Plan (CERP) is charged with restoring the River of Grass and reviving habitat for more than sixty threatened and endangered species, restoring the Everglades National Park’s natural water flows from Lake Okeechobee to Florida Bay, and establishing a reliable environmental, urban, and agricultural water supply while providing improved flood protection. The CERP mission is:

Restoration of America’s Everglades is the world’s largest environmental project of its kind

CERP was designed to capture, store and redistribute fresh water previously lost to tide and to regulate the quality, quantity, timing, and distribution of water flows. The 30-year, $8 billion CERP is being funded, managed, and implemented through a unique 50-50 partnership between the state and federal governments. Situated at a central point at the head of the Everglades, the Everglades Agricultural Area (EAA) Reservoir A-1 has been described as a keystone to the success of CERP by allowing the necessary control of water with a flexible delivery schedule.

1.1.2 Central & Southern Florida (C&SF) Project

CERP provides a framework and guide to restore, protect, and preserve the water resources of central and southern Florida, including the Everglades. It covers 16 counties over an 18,000-square-mile area, and centers on an update of the Central & Southern Florida (C&SF) Project.

The C&SF Project is a multi-purpose project which was first authorized in 1948 to provide flood control, water control, water supply, and other services to the area that stretches from Orlando to Florida Bay. The C&SF Project has performed its function well. At present the C&SF project includes 1,000 miles of canals, 720 miles of levees, and several hundred water control structures. It provides water supply, flood protection, water management and other benefits to south Florida.

However, the C&SF Project has had unintended adverse effects on the unique and diverse south Florida ecosystems, including the Everglades and Florida Bay. While providing flood protection and water supply, it has significantly altered the Everglades and the rest of south Florida's ecosystem. One result of this alteration is that billions of gallons of water that flow through the C&SF project's canal system are wasted. Historically, the rain that fell on south Florida was stored in the system - on and above the ground. Now the C&SF Project quickly drains rainfall off the land. Similarly, during dry periods water is diverted to meet other water supply needs.

The resulting ecological problems are complex. However, in short, the Everglades are not receiving the proper quantity or quality of water at the right place or the right time. Too much or too little water is often sent to the Everglades marshes, coastal estuaries, and Biscayne and Florida bays. The historical and present flow patterns are illustrated in Figure 1.1-1. The Figure also shows the aspiration for the future.

The Water Resources Development Act (WRDA) in 1992 and 1996 provided the U.S. Army Corps of Engineers (USACE) with the authority to re-evaluate the performance and impacts of the C&SF Project, to recommend improvements and or modifications to the project in order to restore the south Florida ecosystem, and to provide for other water resource needs. The purpose of the C&SF Project Comprehensive Review Study (Restudy) was to re-examine the C&SF Project and determine the feasibility of modifying the project to improve the sustainability of South Florida. Specifically, the Restudy investigated:

- Structural and operational modifications to the C&SF Project for improving the quality of the environment
- Improving protection of the aquifer
- Improving the integrity, capability, and conservation of urban and agricultural water supplies
- Improving other water-related purposes

Both the problems with declining ecosystem health and the solutions to Everglades's restoration can be framed by four interrelated factors: quantity, quality, timing, and distribution of water. CERP was designed to capture, store and redistribute fresh water previously lost to tide and to regulate these four key elements.

The principal goal of restoration is to deliver the right amount of water, of the right quality, to the right places, and at the right time. The natural environment will respond to these hydrologic improvements, and health will be restored to the Everglades ecosystem.

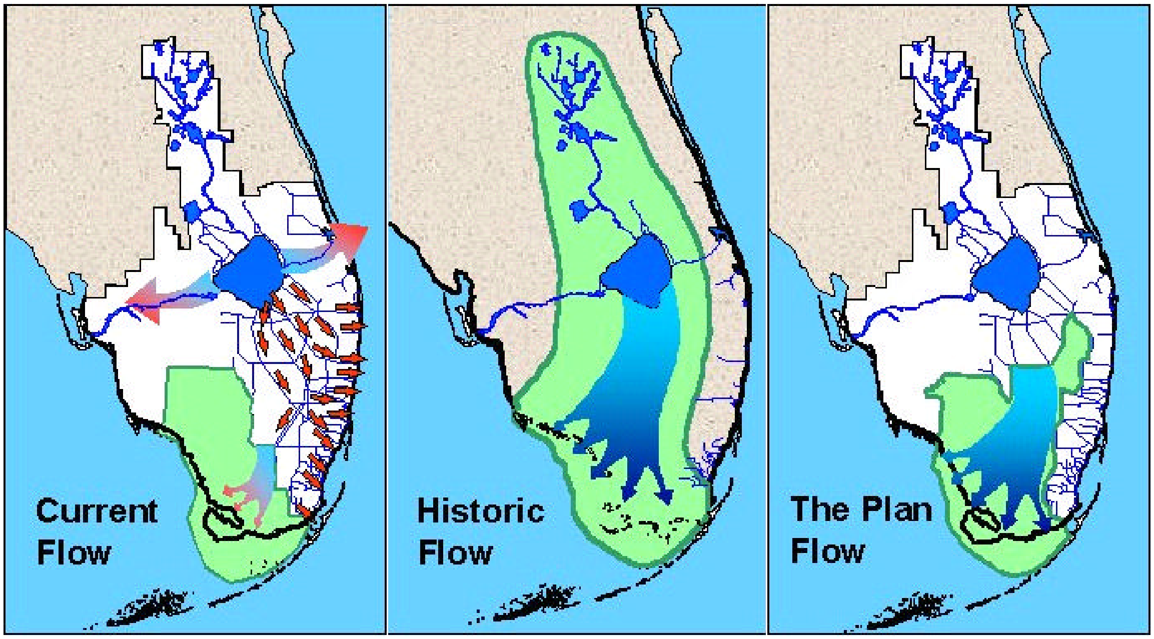
1.1.3 CERP

On December 11, 2000, the WRDA 2000 was signed into law by the President of the United States (Public Law No. 106-541, of the 106th Congress). Title VI, Section 601, of WRDA 2000, describes authorizations specific to CERP. CERP includes more than 60 elements, will take more than 30 years to construct, and will cost an estimated $7.8 billion. CERP's major components are:

1. *Surface Water Storage Reservoirs*
2. *Water Preserve Areas*
3. *Management of Lake Okeechobee as an Ecological Resource*
4. *Improved Water Deliveries to the Estuaries*
5. *Underground Water Storage*
6. *Treatment Wetlands*
7. *Improved Water Deliveries to the Everglades*
8. *Removal of Barriers to Sheetflow*
9. *Storage of Water in Existing Quarries*
10. *Reuse of Wastewater*
11. *Pilot Projects*
12. *Improved Water Conservation*
13. *Additional Feasibility Studies*

CERP's predominant feature is water storage. CERP captures most of the average 1.7 billion gallons of water a day discharged to the ocean. This water will be stored in more than 217,000 acres of new reservoirs and wetlands-based treatment areas, and about 300 underground Aquifer Storage and Recovery (ASR) wells. These features vastly increase the amount of water storage available in south Florida. CERP will ensure a reliable, adequate supply of fresh water for environmental, urban, and agricultural users. Environmental deliveries to the Everglades will meet the 10 parts per billion (ppb) limit for phosphorus and the flow pattern across the Everglades will match the historic pattern more closely as shown on Figure 1.1-1. Approximately 80 percent of the new water captured by CERP will go to the environment and 20 percent will be used to enhance urban and agricultural supplies.

Figure 1.1-1 Restoring the Everglades' Flow Pattern



Florida is a low-lying, flat, rainfall rich state, and is prone to flooding. Today, the C&SF Project provides flood protection on a regional basis for south Florida, supported by many locally operated canal networks. CERP will maintain, and potentially improve, this important flood protection element of the C&SF Project.

Without intervention, the region will experience continued degradation (frequent water shortages) of the Everglades, coastal estuaries, fisheries, and other natural resources; and more frequent flooding.

Implementation of CERP will result in the recovery of healthy, sustainable ecosystems in south Florida. It is a plan that will lead to a strong economy and a much-improved environment for people, and the plants and animals that depend on the natural system for their survival. CERP contains components essential to achieving this goal.

1.1.4 Project Assurances

The C&SF Project provides economic benefits through regional water supply, flood protection, navigation, and recreation. While most people recognize the need for a healthy ecosystem to support the region's economy and jobs, there are others who are concerned that potential restoration projects will displace farms and other businesses, limit development, and reduce job opportunities. By contrast, continued degradation of the south Florida environment will inevitably adversely affect the tourism and recreational industry that are important to the regional economy.

Public concerns about water supply and flood control generally center on preservation of existing forms of protection from relatively frequent flooding, and delivering water for aquifer recharge, as provided by the C&SF Project.

Implementation of CERP will require the cooperation and collaboration of federal, state, and tribal entities. These interests all seek assurances that they will receive the anticipated benefits from CERP. Within WRDA 2000, Congress approved CERP's objectives to restore, preserve, and protect the south Florida ecosystem while providing for water-related needs of the region. Provisions in Section 601(h) of WRDA 2000, "Assurance of Project Benefits," resulted in an agreement between the federal government and the State of Florida.

Section 601(h)(4) of WRDA 2000 specifies that an Integrated Project Implementation Report (PIR) will be used to document consistency with CERP to:

- Satisfy the programmatic regulations
- Comply with the National Environmental Policy Act (NEPA)
- Identify the appropriate quantity, timing, and distribution of water dedicated and managed for the natural system
- Comply with water quality standards and permitting requirements
- Identify the amount of water to be reserved or allocated for the natural system necessary to accomplish the quantity and quality objectives
- Be based on the best available science
- Include an analysis of cost-effectiveness and engineering feasibility of the Project

Section 601(h)(5) of WRDA 2000 specifies a savings clause that must be considered when implementing a project under CERP. Protection of existing legal sources from elimination or transfer and protection of the level of service of flood protection existing as of December 2000 is required by the federal law. The PIR will contain the analyses required to determine whether an elimination or transfer has occurred as a result of implementation of CERP and whether levels of service for flood protection will be reduced.

Implementation of CERP must also be consistent with state law. As a local sponsor, South Florida Water Management District (SFWMD) has responsibilities that are outlined in Section 373.1501(5) of the Florida Statutes (F.S). Subsection (d) requires the SFWMD to provide reasonable assurances that:

- The quantity of water available to existing legal users shall not be diminished by implementation of project components so as to adversely impact existing legal users
- Existing levels of service for flood protection will not be diminished outside the geographic area of the project component
- Water management practices will continue to adapt to meet the needs of the restored natural environment

1.1.5 Phosphorus

Phosphorus pollution has been a concern in the Everglades for many years. It is generally thought to be caused by natural leaching, urban runoff, and agricultural runoff from sugar plantations, vegetable farms, and livestock operations. Excess phosphorus can cause imbalances in vegetation and habitat, and alternative ecosystems. Much of this phosphorus is discharged in water from the EAA. The EAA has been used intensively for farming, particularly sugar cane, since the 1950s. In 1988, the federal government sued the State of Florida and two of its agencies, alleging that water released onto federal lands from agricultural sources contained elevated levels of phosphorus and other nutrients in violation of state water quality standards. Based on a 1992 Consent Decree settling this lawsuit, Florida enacted the Everglades Forever Act (EFA) in 1994. Florida later amended the 1994 EFA to create flexibility in meeting deadlines for phosphorus mitigation to 2016 or later. In July 2003, Florida issued a rule establishing a phosphorus limit of 10 ppb in water released to Federal lands and methods to measure compliance with that limit.

Total phosphorus concentrations are highest in the northern Everglades and lowest in the southern Everglades. This is indicative of phosphorus-rich water in the canals that carry water from the EAA, although urban runoff has also been identified as a contributor.

When bodies of water experience an excessive inflow of nutrients such as phosphorus there is a subsequent increase in plant growth. When the plants die and decompose, they consume dissolved oxygen from the water. If dissolved oxygen levels fall substantially and rapidly, fish and aquatic plant populations suffer. The process also encourages the growth of plants which enjoy these high levels of nutrients. For example, an excessive phosphorus levels in the Everglades is the primary factor behind the conversion of native sawgrass marshes and sloughs to vegetation stands dominated by cattails. This shift in vegetation has resulted in fewer habitats for wading birds and other wildlife, and reduced populations of several native plant species. Further, the rapid growth of cattails is partly responsible for clogging waterways and altering the hydrology in parts of the Everglades.

Excessive phosphorus inflow into the Everglades can be traced back to the 1940s with the clearing of exposed soils, which began to erode and leach phosphorus into waterways that connected to the Everglades. Production intensified after the Cuban revolution in 1959, as Cuban exiles fled to Florida and established sugar plantations. By the mid-1960s, Florida sugar production had increased four-fold. Today, sugarcane production contributes two-thirds of the economic production of Everglades's agriculture, and uses nearly 80 percent of the crop land in the EAA. Sugar production contributes phosphorus to the ecosystem primarily through fertilizers and to a lesser extent through decomposition of plants. Fertilizers and plant decomposition are also the main cause of phosphorus leaching from vegetable production.

Stormwater Treatment Areas (STAs) are constructed wetlands developed to act as buffers between the EAA and the natural wetlands in the Everglades. The purpose of the STAs is to remove excess nutrients, primarily phosphorus, from the water through a number of biological and physical processes. The clean water is then discharged to the Everglades. The EAA Stormwater Treatment Expansion project will increase the size and enhance the performance of existing STAs. The project will expand STA-2 by 2,000 acres and STA-5 by 2,560 acres. This will further reduce nutrient and other pollutant levels in order to meet the water quality standards for the Everglades.

1.1.6 System Limitations

The existing canal system is used for drainage of local runoff, irrigation of the agricultural areas, and for transport of Lake Okeechobee water releases. Currently, the canal capacity is limited and users have permits to discharge between 0.0315 and 0.063 cubic feet per second (cfs) per acre (equivalent to 3/4-inch and 1.5 inches per day of runoff, respectively) during and after precipitation events. This becomes a localized flooding issue when rainfall occurs. Potential agricultural crop damages could be avoided if the water transport could be better managed. The SFWMD has identified several priority removals that need to be considered as part of this Project, listed in order of declining preference, they are:

- Improvement of local drainage – allow increased and more consistent removals of stormwater from agricultural lands during intense rain events
- Reduction or elimination of Lake Okeechobee backpumping – when the capacity of the canal system is exceeded due to intense rain events, stormwater can be pumped back to Lake Okeechobee. This stormwater contains high levels of nutrients, which impair the water quality in the Lake
- Management of Lake Okeechobee water releases to direct flows to the Everglades or for local irrigation – reduce the amount of water discharged to the estuaries

1.2 EXISTING EAA WATER DISTRIBUTION SYSTEM

1.2.1 General

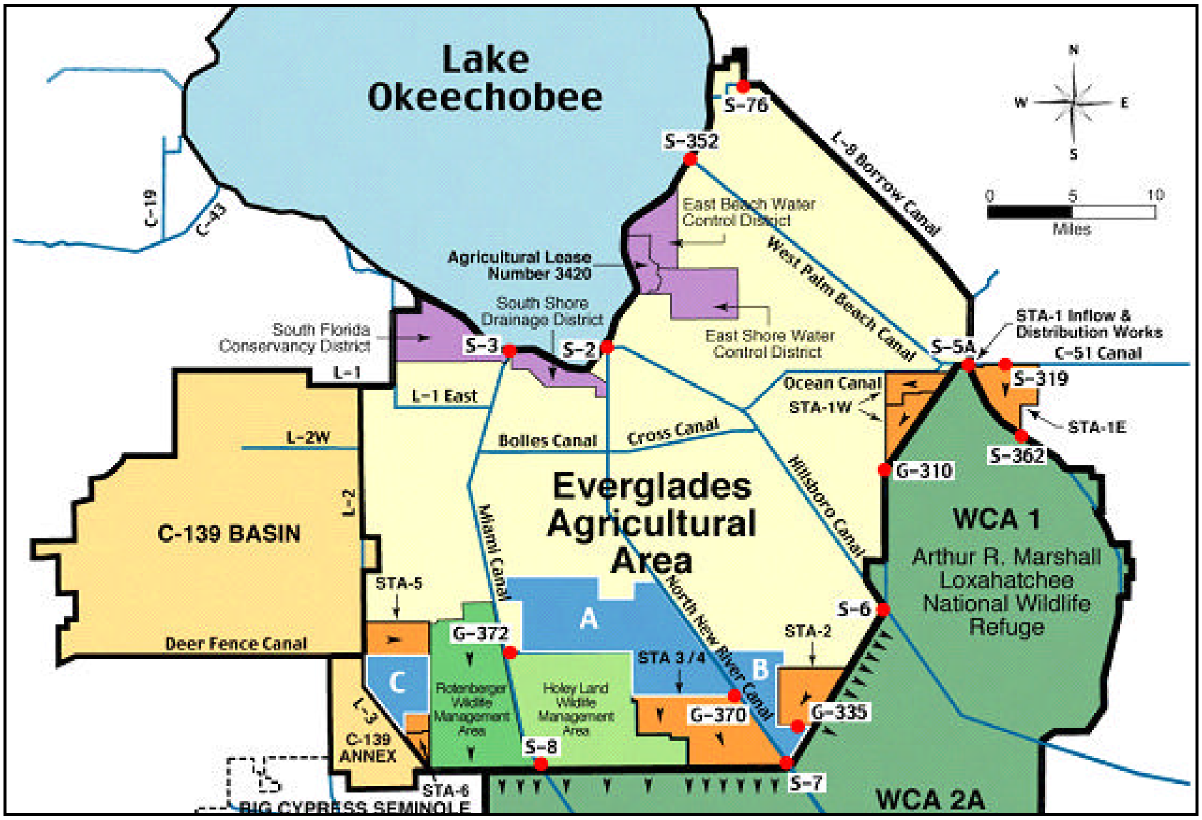
The layout of the existing EAA infrastructure is shown on Figure 1.2-1**Error! Reference source not found.**. In general, the EAA is divided into basins of varying sizes and each basin contributes to a canal or system of canals. The canals are used to convey water into and out of Lake Okeechobee depending on prevailing conditions, operational needs, floods, and irrigation requirements.

At times of need, water for irrigation is released from Lake Okeechobee into the canals running through the EAA for distribution. When drainage is required, the potentially phosphorus rich runoff from the agricultural operations is collected in the canals and pumped to STAs where it is treated before being passed on through the system to the Water Conservation Areas (WCAs) and the Everglades.

However, during intense rainfall events the flow exceeds the capacity of the system to treat the runoff in the STAs, so water has to be backpumped into Lake Okeechobee. This is the role of the S-2 and S-3 pump stations. This backpumping is damaging to Lake Okeechobee, and is a reduction target under CERP.

Changes are being made to the system to enhance STA performance and increase canal conveyance capacity but there is currently no way to manage water flows. Rainfall is pumped when it rains but without storage, stormwater flow can not be regulated for quantity or timing.

Figure 1.2-1 Existing EAA Infrastructure



1.2.2 Configuration of Canals and Basins

The canals provide conveyance from Lake Okeechobee to tide and certain canals allow interbasin transfer within the EAA. The basins are the:

- S-2, S-6, and S-7 Basins, to the East which is associated with the North New River Canal (NNRC) and the Hillsboro Canal
- S-3, S-8 Basins to the West, which is associated with the Miami Canal
- S-5A Basin to the Northeast, which is associated with the West Palm Beach Canal
- Bolles and Cross Canals for interbasin transfer east and west

Lake Okeechobee provides water south to the EAA through three gated spillway structures, S-351, S-354, and S-352.

- S-351 is located next to pump station S-2 and supplies the NNRC and the Hillsboro Canal. The NNRC flows south past the Bolles and Cross Canal confluences to the G-370 pump station, continues on south to structure S-7 and on into the Everglades Protection Area. Currently G-370 pump station feeds the east end of the STA-3/4 Supply Canal. The Hillsboro Canal flows south past the Ocean Canal, then past the Cross Canal confluence. The Hillsboro Canal continues south to structure S-6 and then into the Everglades Protection Area.

- S-354 is located next to pump station S-3 and supplies the Miami Canal which can flow south past the Bolles Canal, down to the G-372 pump station, and then continues south to structure S-8 and on into the Everglades Protection Area. The G-372 pump station pumps water into the STA-3/4 Supply Canal which currently feeds the Holey Land Wildlife Management Area (WMA) and STA-3/4.
- S-352 discharges into the West Palm Beach Canal. The West Palm Beach Canal flows south past the Ocean Canal confluence and into structure S-5A, it then continues east into the C-51 Canal.

The Bolles Canal, Cross Canal, and Ocean Canal lie generally in the east-west direction and connect between basins. The other canals are oriented more north to south. The most common flow pattern is the Bolles Canal flowing east, and the Cross Canal flowing west, both discharging at a common location into the NNRC. There are numerous secondary agriculture canals that connect to the major canals along with seepage ditches common outside the canal levees.

Features and operation of the existing system pertinent to the EAA Reservoir A-1 project are described in more detail below.

1.2.3 S-2, S-6 and S-7 Basin - NNRC

The NNRC is located along the eastern border of EAA Reservoir A-1. It provides drainage to the area extending from gated discharge structure S-351 to pump station S-7. Pump station S-2, situated adjacent to structure S-351, is a 3,600 cfs pump station which currently pumps excess NNRC drainage into Lake Okeechobee.

G-370 pump station is a three-unit pumping station located on the west side of U.S. 27 and the NNRC, about 25 miles south of Belle Glade, Florida. It provides flood protection to the upstream agricultural basins that total approximately 222 square miles. While the removal of stormwater runoff from the upstream basins is the primary function of the pump station, it is also used to convey regulatory releases and supplemental flows from Lake Okeechobee, and water releases sent to STA-3/4 to maintain minimum depth in the treatment cells.

1.2.4 S-3, S-8 Basin - Miami Canal

The Miami Canal carries flows between gated discharge structure S-354 at Lake Okeechobee, and S-8 at the southern edge of the EAA. Pump station S-3 situated adjacent to structure S-354, is a 2,580 cfs pump station that currently pumps excess Miami Canal drainage into Lake Okeechobee.

G-372 pump station is a four-unit pumping station located about 25 miles southwest of Belle Glade, Florida, and 7 miles north of S-8 pump station. It provides flood protection to upstream agricultural basins along the Miami Canal totaling approximately 277 square miles. Similar to the G-370 pump station, the primary function of the pump station is the removal of stormwater runoff from the upstream basins. In addition, it conveys regulatory releases and supplemental flow from Lake Okeechobee and water releases sent to STA-3/4 to maintain minimum depth in the treatment cells. G-372 pump station can also be used to release water into the Holey Land WMA at the SFWMD's discretion.

Normal operations at the Miami Canal system are similar to those of the NNRC.

1.2.5 STA-3/4

The STA-3/4 Supply Canal conveys discharges from G-370 and G-372 pump stations to the inflow structures for the three flow-ways comprising STA-3/4. STA-3/4 is a wetlands treatment system for the removal of phosphorus. The inflow to the northern STA cells is controlled by a series of gated hydraulic structures. Discharge from the STA is controlled by similar structures on the southern side of the area. The treated water from STA-3/4 passes south to WCA-3A.

The STA-3/4 is currently undergoing enhancements, including the construction of an internal levee in Cell 3 with associated structures, small forward pumping stations in Cells 1, 2 and 3, and a Periphyton-Based Stormwater Treatment Area (PSTA) Demonstration Project in Cell 2B (Burns & McDonnell, 2004). These enhancements are scheduled for completion in 2006.

1.3 EAA RESERVOIR A-1 PROJECT

1.3.1 General

In October 2003, the SFWMD decided to pursue a "Dual Track" for the EAA Reservoir Project. While the multi-agency Project Delivery Team, led by the USACE, continues to develop the PIR for the whole of Compartment A, EAA Reservoirs A-1 and A-2 combined, the SFWMD is proceeding with the design and construction of EAA Reservoir A-1 on the land purchased and transferred in a property agreement called the Talisman Exchange.

The EAA Reservoir A-1 Project is an integral element of CERP. Section 601(b)(2)(C)(ii) of WRDA 2000 provides specific authority for implementing the EAA Storage Reservoirs, Phase 1 project. The EAA Reservoir A-1 Project is a sub-component of the Phase 1 project.

The EAA Reservoir A-1 Project is located in western Palm Beach County, generally in Township 46 and Range 37. It is situated in the EAA directly north of STA-3/4, between the NNRC and Miami Canals, and west of U.S. 27. It also adjoins the Holey Land WMA to the southwest.

In accordance with CERP guidelines to capture, store and redistribute fresh water, the EAA Reservoir A-1 facilities will be designed to improve the timing of environmental water supply deliveries to STA-3/4 and the WCAs, reduce Lake Okeechobee regulatory releases to the estuaries, meet supplemental agricultural demands, and increase flood protection within the EAA.

1.3.2 Project Purposes, Goals, Objectives, and Benefits

The purpose of the Phase 1 project, as defined in the EAA Storage Reservoirs Phase 1 Project Management Plan, is to improve timing of environmental deliveries to the WCAs by:

- Reducing damaging flood releases from the EAA to the WCA
- Reducing Lake Okeechobee regulatory releases to estuaries
- Meeting supplemental agricultural deliveries, and increasing flood protection within the EAA

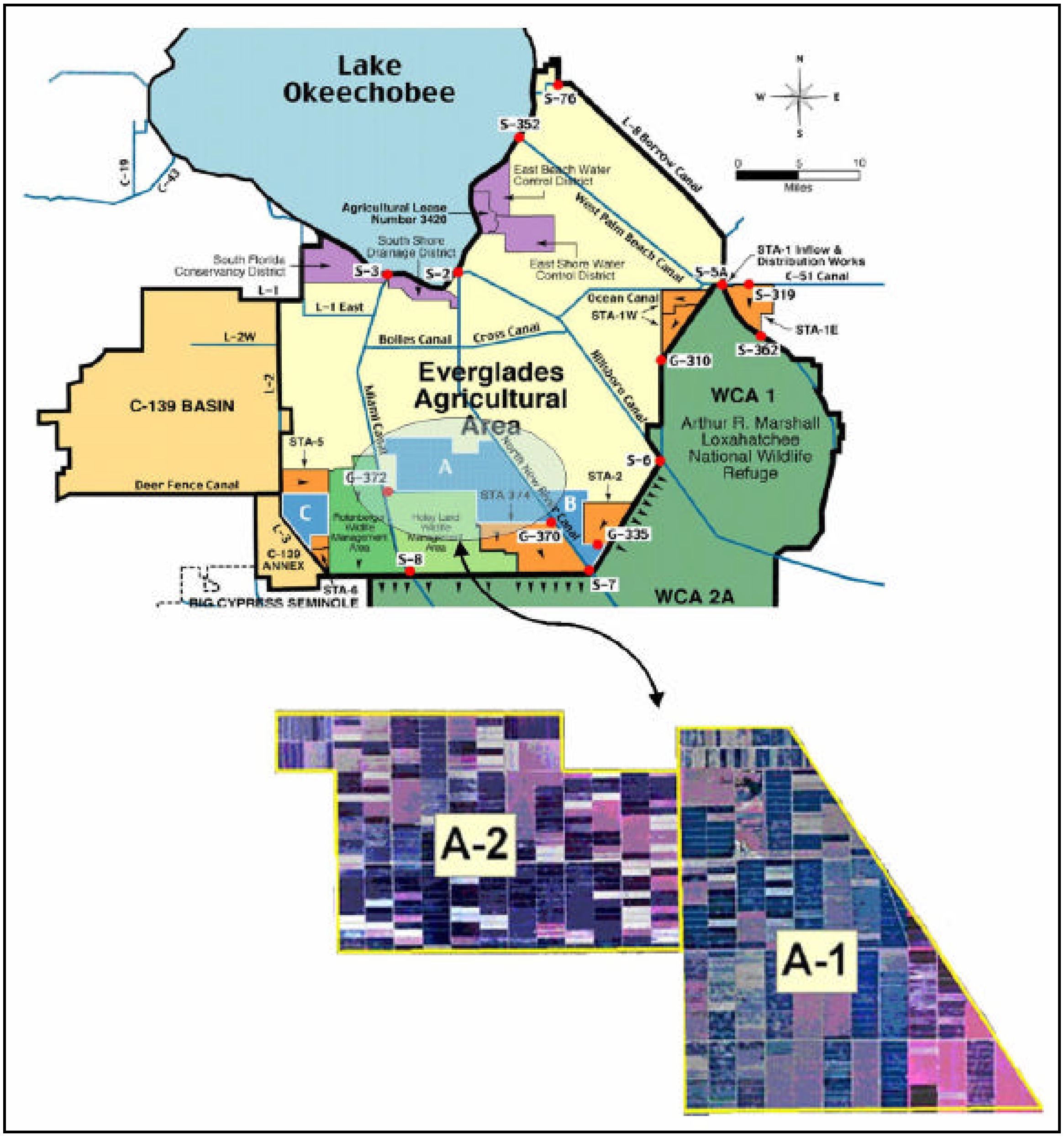
The PIR will provide the most current definition of the purpose and benefits of the EAA Storage Reservoirs Project. Implementation of the EAA Reservoir A-1 Project will meet objectives consistent with the ongoing work by the USACE. In accordance with the USACE's PIR, the objectives of the Compartment A Reservoir include:

- Reduction of the Lake Okeechobee regulatory releases to the estuaries and backpumping from the Study Area (defined as that portion of the EAA that most influences its reservoir site) into Lake Okeechobee by sending the water to the EAA Reservoir A-1
- Improved environmental releases through the storage of water and release to the Everglades during the dry season
- Flow equalization and optimization of treatment performance of STAs by capturing peak storm event discharges within reservoirs for slow release to the STAs
- Improved regional water supply for the agricultural community currently served by the EAA canals and other areas served by Lake Okeechobee

The EAA Reservoir A-1 Project covers approximately 17,000 acres and is designed to store stormwater originating within the S-2/7, S-3/8, S-236 and C-139 basins and releases from Lake Okeechobee, all located generally north of the EAA Reservoir A-1 Project site. A schematic of the EAA Reservoir A-1 and its relationship to the other EAA infrastructure is shown in Figure 1.3-1.

The EAA Reservoir A-1 is one of several reservoirs that are essential in fulfilling CERP's need to "capture, store and redistribute" fresh water. Further, it will improve the "quantity and timing" of delivery of fresh water to meet environmental and agricultural deliveries. Because of its critical place in the overall plan, the EAA Reservoir A-1's implementation was prioritized under the State of Florida's Acceler8 program. Projects in the Acceler8 program are implemented under an accelerated schedule with funding provided by the State of Florida. With the goal of providing maximum benefits for initial investment, the Acceler8 program will provide for construction of the EAA Reservoir A-1, with construction of EAA Reservoir A-2 to follow at a future date.

Figure 1.3-1 EAA Reservoir A-1 and Surrounding Infrastructure



1.3.3 Key Features

The key features of the EAA Reservoir A-1 Project include the following:

- Approximately 190,000 acre-feet of storage with a perimeter embankment and seepage canal
- Northeast pump station that pumps from NNRC (3,600 cfs)
- Connector canal from the NNRC to a new northeast pump station
- Seepage pump stations

- Gated discharge structures
- New four lane bridge on U.S. 27 across the new connector canal

The EAA Reservoir A-1 is intended to store water from the S-2, S-6, and S-7 Basins, collected from the NNRC, and release it to STA-3/4 for treatment before release to WCA-3A.

1.3.4 Plans for Further Development

The EAA Reservoir A-1 Project is the first phase of an ultimate EAA Reservoir Storage System that could store approximately 360,000 acre-feet of water over 30,000 acres of SFWMD-owned land between the NNRC and the Miami Canal. The USACE PIR process is currently evaluating Compartment A, which includes EAA Reservoir A-1 and EAA Reservoir A-2. Currently, only EAA Reservoir A-1 is part of the Acceler8 program. It is anticipated that the construction of EAA Reservoir A-2 will follow in a few years.

In addition, there are several other projects within the Acceler8 program which are all due to be completed by 2011, 11 years ahead of the CERP schedule. They are:

- C-44 (St. Lucie Canal) Reservoir/STA
- C-43 (Caloosahatchee River) West Reservoir
- Bolles and Cross Canals Improvements
- EAA STA Expansion
- Water Preserve Areas - Includes Sites 1, C-9, C-11, Acme Basin B, WCA-3A/3B
- Picayune Strand (Southern Golden Gate Estates) Restoration
- Biscayne Bay Coastal Wetlands - Phase 1
- C-111 Spreader Canal

These Acceler8 projects are designed to contribute as much of the benefits from CERP as quickly as possible. The changing dynamics of the system are modeled by the South Florida Water Management Model (SFWMM) Everglades Construction Project (ECP) 2010 and ECP 2015 simulations (version 5.4.2). This is the same model (including version number) as the ECP 2010 and 2015 runs for the Regional Feasibility Study. The outputs of those simulations have been the foundation of this Basis of Design Report (BODR).

The remainder of the CERP projects will follow as time and resources allow.

1.4 BASIS OF DESIGN REPORT

1.4.1 Purpose and Scope

As part of the BODR, Black & Veatch Corporation (Black & Veatch) evaluated alternative embankment and pumping scenarios for storing 190,000 acre-feet of water on an approximately 16,000 acre tract of land adjacent to the NNRC in the EAA region.

This BODR includes conceptual engineering at a level of detail sufficient to provide specific direction for subsequent preliminary engineering design, final design, and construction phases of the Project. The delivery schedule for the BODR is prior to the completion of the geotechnical and survey phases of the work. This data will be submitted in accordance with schedules for those respective Work Orders.

1.4.2 Authorization

This BODR was authorized under a series of Work Orders issued under Contract CN040932 between SFWMD and Black & Veatch executed July 9, 2004. The Work Orders and their authorization dates are as follows:

Work Order No. 1 Project Initiation...July 30, 2004

Work Order No. 2 Temporary Test (Embankment) Cells – Planning, Design, Construction Observation, Monitoring and Analysis ....November 2, 2004

Work Order No. 3 Wave Run-Up Model..December 3, 2004

Work Order No. 4 Water Balance Modeling ....December 3, 2004

Work Order No. 5 Hydrology and Hydraulics...April 5, 2005

Work Order No. 6 Project Controls and Management...April 6, 2005

Work Order No. 7 Embankment BODR ....April 14, 2005

Work Order No. 8 Surveying...May 25, 2005

Work Order No. 9 Geotechnical Services..May 12, 2005

Work Order No. 10 Pump Station BODR..June 2, 2005

The work authorized by Work Order No. 7, Embankment BODR, and work authorized by Work Order No. 10, Pump Station BODR, are combined herein to provide a single BODR for the EAA Reservoir A-1.

1.4.3 Technical Memoranda Issued

Work on this EAA Reservoir A-1 Project has proceeded on a fast track with many engineering analyses performed in parallel rather than in series. Assumptions were made so that tasks could be initiated without waiting for prerequisite engineering analyses to be completed. As prerequisite tasks were completed, initial assumptions were revisited and either validated or revised. Analyses were documented in numerous Technical Memoranda as listed below. These Technical Memoranda are contained in the Appendices of the BODR.

Work Order No. 2

- Embankment .... Appendix 8-1
- Seepage Control .... Appendix 8-6
- Test Cell Construction and Seepage Monitoring Report .... Appendix 8-9
- Reservoir Seepage Analysis... Appendix 8-10
- Reservoir Configuration .... Appendix 8-11

Work Order No. 3

Work Order No. 4

Work Order No. 5

Work Order No. 7

Work Order No. 10

1.5 REFERENCES

Burns & McDonnell, *Appendix 4B-13: Conceptual Design of a PSTA Demonstration Project in STA-3/4* Everglades Consolidated Report, 2004.

SECTION 2

EXISTING SITE CONDITIONS

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TABLES

FIGURES

2. EXISTING SITE CONDITIONS

The planned EAA Reservoir A-1 Project is located about 16 miles south of Lake Okeechobee in western Palm Beach County, Florida. It is in the Everglades physiographic area, an area of low relief with the natural ground surface of the EAA Reservoir A-1 Project site lying generally between elevations 8 and 11 feet NAVD88.

The EAA Reservoir A-1 site is an agricultural area primarily used for growing sugarcane. The area is drained by a system of canals constructed during the second half of the last century as the Central and Southern Flood Control Project. Pumping and flooding of these canals is used by the local sugar producers to regulate the groundwater level during planting and harvesting of the primary crop (sugarcane). It has a subtropical climate with about 55 inches of rain per year.

2.1 *CLIMATE*

The climate of southeast Florida is characterized as subtropical. The average annual temperature in Palm Beach County, Florida is approximately 73 degrees Fahrenheit. According to the Southeast Regional Climate Center (one of six regional climate centers in the United States directed by the National Oceanic and Atmospheric Administration [NOAA]), the average maximum daily temperature for the Belle Glade Experimental Station (No. 080611) was 83.5 degrees Fahrenheit for the period of record May 1, 1924 through February 29, 2004. The average minimum temperature for the same period of record was 61.7 degrees Fahrenheit. Average total precipitation at the Belle Glade Experimental Station for this period of record was 55.32 inches. The Belle Glade Experimental Station is approximately 10 miles north of the EAA Reservoir A-1 Project site.

Approximately 75 percent of the annual precipitation occurs during the wet season months of June through October. During this season, scattered and isolated convective thunderstorms occur frequently over land. Tropical storms and hurricanes also occur during the wet season and can provide significant rainfall and extreme winds in a short period of time. Rainfall from November through May (the dry season) is usually the result of large frontal systems from the north and are broadly distributed rather than localized. According to the Southeast Regional Climate Center, the wettest average month for the period of record is June (8.52 inches), while the driest average month for the period of record is December (1.71 inches).

2.2 *CURRENT LAND USE*

The EAA was designated by the United States Congress in 1948. It is bounded by Lake Okeechobee on the north and the Everglades National Park on the south. The EAA was created as a result of draining the northern Everglades for agricultural use. It encompasses about 27 percent of the historic Everglades and consists of an area of approximately 700,000 acres of farmland. The major crop in the EAA is sugarcane, but winter vegetables are also grown.

Land use within the approximate 16,000 acre EAA Reservoir A-1 Project was reviewed using the SFWMD Florida Land Use, Cover and Forms Classification System (FLUCCS) as a Geographical Information System (GIS) data layer. Nearly the entire EAA Reservoir A-1

Project site, as well as adjoining lands to the north, northwest, and east, is designated for sugarcane production (FLUCCS 2156). A small rectangular-shaped parcel in the northern portion of the EAA Reservoir A-1 Project site is designated industrial land use (FLUCCS 1500), and was occupied by the Talisman Sugar Corporation processing facility. The Environmental Site Assessments of this land are described in Section 2.7.3 of this report. The Holey Land tract is southwest of the EAA Reservoir A-1 Project site and is designated as freshwater marshes with sawgrass. The southern adjoining property is occupied by STA-3/4.

2.3 *GEOLOGY AND SOILS*

2.3.1 Regional

The EAA Reservoir A-1 Project is located south of Lake Okeechobee within the Everglades physiographic subdivision of the Southern Zone (White, 1970). The Everglades is generally a flat, geologic depression between the Immokalee Rise and Big Cypress Spur physiographic subdivisions on the west, and the Atlantic Coastal Ridge physiographic subdivision on the east. The Everglades extends southward from Lake Okeechobee to Florida Bay with elevations near sea level. With the exception of the EAA, the Everglades landscape consists primarily of sawgrass marsh with hammocks of willow, myrtle and bay trees.

The United States Department of Agriculture, Natural Resources Conservation Service (NRCS and formerly known as the Soil Conservation Service) published a soil survey for the Palm Beach County area in the mid 1970s (McCollum et al., 1978). Seven primary soil types were identified in the EAA region as Torry muck, Terra Ceia muck, Pahokee muck, Lauderhill muck, Dania muck, Okeelanta muck, and Okeechobee muck. The soils at EAA Reservoir A-1 include the Pahokee muck (primarily in the southern portion of the site) and Lauderhill muck (primarily in the northern portion of the site). Based on borings at the EAA Reservoir A-1 Project site, the muck ranges in thickness from less than 1 foot to approximately 5 feet.

According to the NRCS, the soils located beneath the former Talisman Sugar Corporation processing facility are classified as Urban land. Urban land soils are those which have been disturbed due to development.

The generalized regional geologic/hydrogeologic conditions for Palm Beach County are provided in Figure 2.3-1. It should be recognized that this representation is not all inclusive since the geology in southeast Florida is very complex, particularly the geology of the Pliocene-Pleistocene to Holocene Epochs. However, the primary geologic and hydrogeologic units that are formally recognized in Palm Beach County are represented.

In general, the surface and near surface geology of the region is complex and ranges from unconsolidated, variably calcareous and fossiliferous quartz sands to well indurated, sandy, fossiliferous fresh and marine limestones (Scott, 2001). These sediments are Pleistocene to recent in age, and blanket most of Palm Beach County except for the Atlantic Coastal Ridge sediments on the east coast. The regional geologic units are generally referred to, in descending order, as the Lake Flirt Marl, Fort Thompson Formation, and Caloosahatchee Formation. The total thickness of these units can range from 50 to nearly 200 feet in the region.

The Pliocene-age Tamiami Formation underlies the Caloosahatchee Formation. The Tamiami Formation contains a wide range of mixed carbonate-siliciclastic lithologies and associated faunas (Missimer, 1992). The Tamiami Formation in the area is approximately 100 feet thick.

The upper portion of the Tamiami Formation and overlying geologic units comprise the surficial aquifer system in Palm Beach County.

The Miocene-age Hawthorn Group underlies the Tamiami Formation. The Hawthorn Group consists of an interbedded sequence of widely varying lithologies and components that includes limestone, dolomite, dolosilt, shell, quartz sand, clay, phosphate grains and mixtures of these materials (Reese and Memberg, 2000). The characteristics that distinguish the Hawthorn Group from underlying units are its high and variable siliciclastic and phosphatic content; its color, which can be green, olive-gray, or light gray; and its gamma-ray log response. According to Scott (1988), the Hawthorn Group is approximately 700 feet thick in the region. The Hawthorn Group sediments retard the exchange of groundwater between the overlying surficial aquifer system and the underlying Eocene-age carbonates of the Floridan aquifer system, and is hydrogeologically referred to as the intermediate confining unit.

The Eocene-age carbonates underlying the Hawthorn Group include, in descending order, the Ocala Limestone, Avon Park Formation, and Oldsmar Formation. The overlying Oligocene-age Suwannee Limestone is thin to discontinuous in the EAA region, and likely not present in the east half of Palm Beach County (Miller, 1986). The cumulative thickness of the Eocene-age carbonates in the region is approximately 2,500 feet (Miller, 1986).

Figure 2.3-1 Generalized Regional Geology and Hydrogeology

Series	Geologic Unit	Approximate thickness (feet)	Lithology	Hydrogeologic unit	Approximate thickness (feet)
HOLOCENE	PAMLICO SAND	0-50	Quartz sand with shelly intervals	SURFICIAL AQUIFER SYSTEM	150-380
PLEISTOCENE	ANASTASIA FORMATION	0-200	Quartz sand, shell, and coquina		
	FT. THOMPSON FORMATION	0-40	Alternating marine molluscan limestone and freshwater marl		
PLIOCENE	TAMIAMI FORMATION	0-200	Sandy, shelly limestone, calcareous sandstone, quartz sand, and clayey sand		
MIOCENE AND LATE OLIGOCENE	HAWTHORN GROUP	600-800	Clay, marl, dolosilt, micritic limestone, clayey sand, silt, and phosphate grains	INTERMEDIATE CONFINING UNIT	600-700
	MARKER UNIT	90-130	Micritic limestone to marl, chert nodules, some phosphate grains		
?	BASAL HAWTHORN UNIT	30-355	Limestone, dolomite, shell, sand, sandstone, and calcareous clay or silt, abundant phosphate grains in places	FLORIDAN AQUIFER SYSTEM — LOWER HAWTHORN PRODUCING ZONE	10-180
EARLY OLIGOCENE	EOCENE GROUP — SUWANNEE LIMESTONE	0-150	Fossiliferous, calcarenitic limestone	UPPER FLORIDAN AQUIFER	500-700 ?
EOCENE — LATE	OCALA LIMESTONE	0-300	Chalky to fossiliferous, calcarenitic limestone		
EOCENE — MIDDLE	AVON PARK FORMATION	900-1,200	Fine-grained, micritic to fossiliferous limestone and dolomite	MIDDLE CONFINING UNIT	0?-900
	DOLOMITE UNIT	0-900			
EOCENE — EARLY	? OLDSMAR FORMATION	? 1,100-1,500	Fine-grained, micritic to fossiliferous limestone, dolomitic limestone, and dense dolomite	LOWER FLORIDAN AQUIFER; BOULDER ZONE	1,800; 300-650
PALEOCENE	CEDAR KEYS FORMATION	500-600	Dolomite and dolomitic limestone		
		1,500?	Massive anhydrite beds	SUB-FLORIDAN CONFINING UNIT	1,500?

From Hydrogeology and the Distribution of Salinity and the Floridan Aquifer System, Palm Beach County, Florida WRIR 99-4061 2000, USG

2.3.2 Site

The EAA Reservoir A-1 Project site has been investigated in a progressive sequence of borings spaced throughout the site area. One hundred forty-five borings were completed for the South Florida Water Management District around the reservoir perimeter in 2003 and early 2004. Twenty borings to a depth from 50 to 100 feet below ground surface (bgs) were completed at the EAA Reservoir A-1 Project Test Cell site for the Test Cell Project design in December 2004, and an additional eight borings were completed during the Test Cell construction in early 2005.

The borings generally penetrated through about 1/2 to 2 feet of surficial peat/muck and marl, then through 22 to 26 feet of primarily carbonate sand and limestone, and then into primarily shelly quartz sand with sparse limestone to their completed depths. The marl beneath the peat and muck is known by some authors as the Lake Flirt Marl (Reese and Cunningham, 2000; Harvey et. al., 2002), but is undifferentiated from the peat and muck layer for this report. The upper carbonate sand and limestone constitutes the Fort Thompson Formation at the site. Below this, the shelly sand and sparse limestone constitutes the Caloosahatchee Formation and possibly part of the Tamiami Formation.

The top of the Fort Thompson Formation consists of a limestone layer about 4.5 to 5 feet thick, which is locally called caprock. The caprock is generally white, light gray, tan or yellowish brown with variable amounts of weathering; it is occasionally fractured and contains voids and inconsistencies. The caprock is underlain by a silty carbonate sand extending to about 23.5 to 24.5 feet deep, where another hard limestone layer 1.5 to 3 feet thick is encountered. A thinner, hard limestone layer about 1/2 to 1 foot thick is often encountered at around 16 to 17 feet deep. The sand and lower limestone layers are generally white to very pale brown. Laboratory testing of the sand sampled in the borings averaged 84.2 percent calcium carbonate content with an average of 22 percent passing the #200 sieve in gradation tests. Visual inspection of the sand samples from the borings reveals that they include shell fragments, and tend to be angular and platy.

All the Fort Thompson Formation limestone layers exposed in core or in excavations at the EAA Reservoir A-1 Project site are very fossiliferous. The sand exposed in the seepage collection canals and dewatering sumps was abundantly fossiliferous with gastropods, pelecypods, corals, and echinoderms.

The top of the Caloosahatchee Formation is composed of fine grained, subrounded, shelly quartz sand that is mixed with shelly carbonate sand similar to that in the Fort Thompson Formation. The Caloosahatchee Formation at the site is 30 to 60 feet thick; however, the interface between this formation and the underlying Tamiami Formation is difficult to define. The proportions of carbonate to quartz sand vary. Laboratory testing on the sampled sand indicated an average calcium carbonate content of 30.1 percent, and an average 12.1 percent of material passing the #200 sieve. The primary color of the geologic material in the Caloosahatchee Formation is light greenish gray.

Other geologic information may indicate that the Caloosahatchee Formation is not present at the EAA Reservoir A-1 Project site. For instance, recent geological work (Reese and Cunningham, 2000) has redefined the stratigraphy of the area. Presently, the Tamiami Formation has several recognized named and unnamed geologic members including the Ochopee Limestone Member and the Pinecrest Sand Member. Both Tamiami Formation members contain sandy strata, but the Pinecrest Sand Member is principally shelly, fine grained, quartz sand. The sands in the Caloosahatchee and Tamiami Formations are generally differentiated based on the fossil assemblages observed in outcrops, but key indicator fossils are typically not recovered in borings (Scott, 2005). Therefore, interpretation of the contact between the Caloosahatchee Formation and Tamiami Formation at the EAA Reservoir A-1 Project site is not possible.

2.4 *SEISMICITY*

The Uniform Building Code Seismic Zone Map (USACE 1995), shows that the entire state of Florida is in seismic Zone 0. No capable faults or recent earthquake epicenters are known to exist near the project site.

SFWMD's requirements for seismic evaluation of CERP high hazard potential dam projects, such as EAA Reservoir A-1, are described in DCM-6. Although Southern Florida is a low seismicity region, the possibility exists for earthquake imposed seismic loads on project structures. The potential earthquake loading is low enough that compacted embankments should not be damaged, but the natural sand foundations of the embankments could potentially be affected.

Loose, saturated sandy soils are susceptible to liquefaction (loss of strength from shaking). This loss of strength could lead to sliding or settlement, possibly resulting in embankment failure. DCM-6 presents the design criteria developed jointly by the SFWMD and the U.S. Army Corps of Engineer’s (USACE) for evaluating liquefaction potential of CERP impoundments.

2.5 *SURFACE WATER HYDROLOGY*

Lake Okeechobee provides water south to the EAA through three structures, S-351, S-354, and S-352. Structure S-354 supplies the Miami Canal which flows south past the Bolles Canal, down to the G-372 pump station, and then continues south to structure S-8 and on into the Everglades Protection Area. The G-372 pump station pumps water into the STA-3/4 Supply Canal which currently feeds the Holey Land WMA and STA-3/4. S-351 supplies the NNRC and the Hillsboro Canal. The NNRC flows south past the Bolles and Cross Canals to G-370 pump station, continues on south to structure S-7 and on into the Everglades Protection Area. The NNRC will be used to supply the proposed northeast pump station located at the northeast end of the EAA Reservoir A-1. G-370 pump station may also be reconfigured to pump into the EAA Reservoir A-1. Currently it feeds the east end of the STA-3/4 Supply Canal. The Hillsboro Canal also is supplied from structure S-351 where the discharge flows south past the Ocean Canal, then past the Cross Canal. The Hillsboro Canal continues south to structure S-6 and then into the Everglades Protection Area. Structure S-352 discharges into the West Palm Beach Canal. The West Palm Beach Canal flows south past the Ocean Canal and into structure S-5A, it then continues east into the C-51 Canal.

The Bolles and Cross Canals flow east-west in direction and water movement can be in either direction. However, the most common flow pattern is the Bolles Canal flowing east, and the Cross Canal flowing west, both discharging at a common location into the NNRC. There are numerous secondary agriculture canals that connect to the major canals along with seepage ditches common outside the levees. The secondary agriculture canals are responsible for north-south water movement.

The NNRC is the source for the G-370 pump station and will be the source for the proposed northeast pump station. Its major sources of water are from Lake Okeechobee, Bolles Canal, Cross Canal, and local farm permitted pump discharges. The local farm pump discharge locations south of Bolles and Cross Canals, and north of the G-370 pump station have been consolidated to nine locations for computer modeling as lateral inflow points into the NNRC. South of Bolles and Cross Canals and north of the proposed northeast pump station are seven locations. For a runoff event of 3/4 inches these pumps are assumed to discharge a total of 1,016 cfs. The remaining two farm pump locations are between the proposed northeast pump station and the G-370 pump station. For a runoff event of 3/4 inches these two pumps are assumed to discharge a total of 745 cfs.

Modeling of the surface water has been conducted by the Office of Modeling using the 2x2 model of the EAA (ECP 2010 and 2015). This is described in Section 6.2.

The construction of the EAA reservoir will require that the embankment be designed to withstand wind and precipitation design conditions. Four wind and precipitation design conditions were identified in draft form in DCM-2 (Haapala Et Al., 2005a). The conditions were 1) 100 year wind with probable maximum precipitation, 2) category five hurricane with 100-year storm, 3) probable maximum wind (200 mph), and 4) a storm specific wind and precipitation (Hurricane Easy). These conditions are described in Section 5.3.

2.6 *GROUNDWATER HYDROLOGY*

Since the intermediate confining unit is located approximately 200 to 250 feet bgs and will restrict any seepage from EAA Reservoir A-1 that might reach this depth, only the surficial aquifer system lying above the confining unit is of concern for this project. With the high degree of communication between groundwater and surface water in the area, the groundwater gradient in the surficial aquifer system is controlled to a large extent by the operation of the hundreds of canals throughout the region. Therefore, even though the general regional gradient in the surficial aquifer system is believed to be southward, localized gradients may actually be in other directions in portions of the area surrounding the EAA Reservoir A-1 site due to the operation of canals and wells in the region. Future seepage from the EAA Reservoir A-1, the operation of the seepage canal, and modifications to the operation of the NNRC will also change these groundwater gradients in the surficial aquifer system near the EAA Reservoir A-1.

To interpret the groundwater pressure profile in the surficial aquifer system when seepage occurs, a series of more than 70 piezometers were installed for the Test Cell Project. The piezometers were constructed in nested sets and screened at various depths. The pressure readings from the piezometers were used to determine both the horizontal and vertical hydraulic conductivity (K_h and K_v, respectively) for each of the geologic units comprising the surficial aquifer system using both three-dimensional and two-dimensional groundwater models. Based on previous studies and geologic data collected from site specific borings, five separate units were identified for evaluation of hydraulic conductivity. The five layers (in descending order) are: (1) muck/peat and marl, (2) caprock, (3) the Fort Thompson Formation, (4) the Caloosahatchee Formation, and (5) the Tamiami Formation. The K_h and K_v values (derived by calibrating each of the groundwater models to the measurements taken during the Test Cell Project) are shown in Table 2.6-1. The slight differences between the calibrated K_h and K_v values are because of inherent differences between the two models. When applied to the Test Cell Project, the two sets of K_h and K_v values produce very comparable results. In addition, the

USACE has prepared a three-dimensional groundwater model of the EAA Compartment A area. Throughout the groundwater modeling effort, there was significant coordination with the USACE, which will continue to occur through the design process.

Table 2.6-1 Hydraulic Conductivity Values Determined by Test Cell MODFLOW Model

Layer	K_h (feet/day)	K_v (feet/day)
Muck/peat and marl[1]	100	100
Caprock	500	1.1
Fort Thompson Formation	400	10
Caloosahatchee Formation	400	8
Tamiami Formation [2]	36	18

1 Muck was removed from Test Cells, so calibration of the K values for the muck was not possible. The listed values were determined by the United States Army Corps of Engineers (USACE) through laboratory/field testing of the muck which were K_h = 40 feet/day and K_v = 9 feet/day (USACE, 2005). These values were increased as shown to account for the significant area where muck does not exist (Seepage Evaluation, Groundwater Model Memorandum, Black & Veatch, July 11, 2005).

2 The Test Cell piezometers did not penetrate the deeper portions of the surficial aquifer system, so calibration to the published K values for the Tamiami Formation was not possible. The above conductivities reflect the USACE's values determined from laboratory/field testing.

2.7 *ENVIRONMENTAL CONDITIONS*

Under CERP, an area of land called Parcel A in the center of the EAA south boundary was designated as the site for a storage reservoir. Parcel A consists of about 30,000 acres including approximately 583 acres of open water, 97 acres of shrub and brushland, 206 acres of wetlands, and the remaining acres in agricultural use. Under the Acceler8 program, Parcel A was divided into two portions: A-1 and A-2. EAA Reservoir A-1 is approximately the Eastern half of Parcel A over an area of close to 17,000 acres. Historically, the project area was predominantly sawgrass marsh but in the mid-1900s it was drained for agricultural production.

The discussion of environmental conditions focuses on two specific issues: (1) vegetation and wetlands and (2) endangered species. A summary of the information follows and more detailed information is contained in Appendix 2-1. Environmental Site Assessments are described in Section 2.7.3.

2.7.1 Vegetation and Wetlands

2.7.1.1 Existing Conditions

The EAA Reservoir A-1 Project area is covered by two soils that are generally organic sediments, Pahokee muck (Depressional), and Lauderhill muck (Depressional). In turn, these soils are underlain by soft, porous limestone bedrock. A desktop survey of the area was conducted by the U.S. Fish and Wildlife Service (USFWS) that identified 81 potential wetland areas. Aerial surveys were used by an interagency team of biologists from the United States Fish and Wildlife Service (USFWS), USACE, SFWMD, United States Environmental Protection Agency (EPA), Florida Fish & Wildlife Conservation Commission (FWC), and

Florida Department of Environmental Protection (FDEP) for verification of the potential wetland areas. Of the 81 potential wetlands, five wetlands were verified, totaling 205.88 acres. The habitat quality of the five verified wetlands was determined as one Category 1 wetland (13.07 acres), one Category 3 wetland (1.73 acres), one Category 5 wetland (3.45 acres), and two Category 6 wetlands (187.63 acres).

USFWS (Slack, 2005) issued a Planning Aid Letter (PAL) to USACE on March 11, 2005, in which they provided guidance and recommendations on resource conservation issues for the EAA Reservoir Storage Project. USFWS recommended including a habitat buffer on the north and west sides, and littoral shelves along the seepage canals and on the internal sides of the embankment. USFWS recognized that littoral shelves on the interior sides of the EAA Reservoir A-1 may be cost-prohibitive. These recommendations are addressed in the discussion on the EAA Reservoir A-1 configuration in Section 12.

2.7.1.2 Potential Impacts

Due to the presence of extensive sugarcane farming and limited acreage of natural habitats on the EAA Reservoir A-1 Project site, adverse effects to native vegetation are limited to wetland areas. As a result of the proposed EAA Reservoir A-1 Project, approximately 206 acres of emergent and scrub-shrub wetland will be converted to open water aquatic habitat. All impacts to upland areas are to lands in active agricultural use.

The existing wetlands (205.88 acres) within the EAA Reservoir A-1 Project are considered to be disturbed wetlands due to the sugarcane farming practices that comprise the majority of the surrounding area. Most of the wetlands are dominated by nuisance and/or exotic vegetation as identified by the Florida Exotic Pest Plant Council on the List of Invasive Species and appear to be isolated and surrounded by sugarcane farming. Although the habitat is predominately exotic, the wetlands still provide habitat and foraging for medium and small sized animals. The wetlands also provide water storage and promote water quality.

The proposed EAA Reservoir A-1 will replace the wetland habitat with an aquatic habitat that will be approximately 16,000 acres in size. The area is projected to re-vegetate through natural recruitment with aquatic plants and wetland plants particularly around the edge of the water. The aquatic habitat will provide habitat and foraging for medium and small sized mammals, reptiles, amphibians, birds, fish and invertebrates. The increase in open water will specifically provide an optimal location for migratory birds for habitat and foraging, and increased utilization by fish and other aquatic species. The water storage function will increase due to the large capacity of the EAA Reservoir A-1. There will be deep water refugia that will be approximately five percent of the total acreage. The EAA Reservoir A-1 will also provide a filter to “polish” water, improving water quality.

Littoral benches along seepage canals (approximately 48 acres) will also be constructed around the exterior of the embankment. There may be intermittent littoral shelves within the canals, depending on the characteristics of the cap rock at specific locations. These littoral shelves will depend on natural vegetative recruitment from surrounding seed sources. The littoral shelves will also provide habitat and foraging for a variety of species, as well as water storage and increased water quality.

Using Chapter 62.345, Florida Administrative Code (F.A.C.), the Uniform Mitigation Assessment Method (UMAM) adopted by the State of Florida and accepted for use by the

USACE, has determined that the construction of the proposed EAA Reservoir A-1 will more than adequately off-set the impact to the wetlands on-site and address any mitigation requirements many times over (33 Functional Loss Units: 10,560 Relative Functional Gain Units).

The relative functional gain is much higher than the functional loss due to the relative size between the existing wetlands and the proposed EAA Reservoir A-1, and the low quality habitat of the existing wetlands. The mitigation required to offset proposed impacts is 0.0031 acres.

In summary, the proposed 16,000 acre EAA Reservoir A-1 will provide greater wetland functions than the current conditions on-site, and will act as more than enough mitigation for the flooding impacts to the 205.88 acres of existing wetlands. The seepage canals may also provide some net wetland benefit; the amount is dependent on the acres of littoral shelf that can be created, if practicable.

2.7.2 Fish and Wildlife

Prior to the agricultural alterations to this area, wildlife was similar to that found on the adjacent Holey Land WMA. Wildlife species typically seen at the Holey Land WMA include white-tailed deer, common snipe, marsh rabbit, blue-winged teal, mottled ducks, and other waterfowl.

2.7.2.1 Existing Conditions

The proposed EAA Reservoir A-1 Project area is dominated by sugarcane production interspersed with isolated emergent wetlands and drainage canals dissecting the property. The USFWS stated that native habitats for fish and wildlife are not a significant component of the area due to alterations for agriculture that have removed most native vegetation. The quality of habitat provided by the existing canal and wetlands is low. However, these wetland habitats do provide foraging habitat for birds, and the canals provide habitat for fish, reptiles, and invertebrates.

The Florida Natural Areas Inventory (FNAI) was consulted to identify the elemental occurrences of protected species within the EAA Reservoir A-1 Project area, and none was found. Potential habitat for the wood stork and Florida panther was identified southwest of the EAA Reservoir A-1 Project area that is in the Holey Land WMA. The FWC Potential Habitat Model was used by the USFWS to identify and calculate potential habitat areas for those wildlife species that may occur in the EAA Reservoir A-1 Project area. Out of 33 possible species, potential habitat was identified for 14. One of these is federally endangered (wood stork) and two are federally threatened (American alligator and eastern indigo snake).

2.7.2.2 Potential Impacts

Due to the limited natural habitat within the EAA Reservoir A-1 Project area, long-term adverse impacts to fish and wildlife, including state and federal protected species, are not anticipated. Waterfowl, fish, and reptiles may experience temporary impacts due to the elimination of existing agricultural ditches and isolated wetlands. Impacts to all wildlife species can be minimized by gradually flooding the area, thereby allowing the terrestrial wildlife to vacate the area. However, following construction, new habitat will be created that will afford similar foraging opportunities for these species. Potential habitat in the adjacent Holey Land WMA will be impacted indirectly by the control of water levels and improved water quality at the WMA. Additionally, temporary impacts from the noise from construction activities are anticipated.

2.7.3 Environmental Site Assessments

Under the Talisman Exchange, the Talisman Sugar Corporation (Talisman) in conjunction with The St. Joe Company (SJC), conveyed approximately 55,000 acres of land utilized for sugarcane farming and milling to the United States Department of Interior (DOI), the SFWMD and The Nature Conservancy (TNC). The farmland is located in Palm Beach and Hendry Counties, and consists of the Talisman Farm (approximately 36,000 acres) and several smaller, non-contiguous satellite farms.

The southern portion of the Talisman Farm will become EAA Reservoir A-1. The northern portion of the Talisman Farm, along with the satellite farms, has been exchanged by the SFWMD for land owned by other local sugarcane growers in order to secure a contiguous block of land necessary for creation of the EAA Reservoir A-1 and to assist in restoring water quality in the Everglades.

Prior to conveyance of the Talisman property, Dames and Moore (D&M), on behalf of the SFWMD, performed Phase I and Phase II Environmental Site Assessments on the Talisman owned/leased properties. The Environmental Site Assessments were part of the due diligence effort associated with the potential purchase of the property. The Phase I Environmental Site Assessment was performed to identify potential point source areas of concern. A Phase II Environmental Site Assessment was later performed to determine the status of potential constituents of concern (COC) at each of the areas of concern identified in Phase I. It was not within the scope of work of the Phase II Environmental Site Assessment to fully delineate any potential impacts to soil and/or groundwater.

Based on the Phase I and Phase II Environmental Site Assessments, D&M identified 11 areas at which COC were detected in soil, groundwater, sediment, or surface water at concentrations exceeding regulatory cleanup target levels or guidance concentrations. Transference of ownership of each of the Exclusion Areas was deferred until a Site Rehabilitation Completion Order (SRCO) for each Exclusion Area was issued by the FDEP.

The list of Exclusion Areas included:

- Five pump stations
- Two pesticide mix load areas
- A former labor camp and cropduster landing strip
- A former borrow pit/agricultural landfill
- The former sugar processing mill
- The surface water management areas adjacent to the sugar mill

These areas were primarily impacted with organochlorine pesticides (e.g., DDT), petroleum products, and arsenic.

Professional Services, Inc. (PSI) performed assessment and remediation on all of the Exclusion Areas on behalf of Talisman Sugar Corp. and the SJC. The cleanup objectives for each Exclusion Area within the proposed EAA Reservoir A-1 area were based on the proposed end land use for water storage areas. As such, cleanup target levels were chosen to be protective of potential ecological receptors which are likely to inhabit the area once a reservoir is constructed.

Since the cleanup target levels for protection of wildlife for most of the chemicals found on the Talisman property are more stringent than the cleanup standards for human health, a cleanup to ecological standards is also inherently protective of agricultural workers during the interim period prior to EAA Reservoir A-1 construction, and EAA Reservoir A-1 construction workers.

The FDEP has issued SRCOs for the majority of the Exclusion Areas. These parcels can be conveyed immediately with no restrictions. On the remaining parcels, the remediation work has been completed to the satisfaction of the FDEP and the FDEP has issued memoranda of technical concurrence. However, a deed restriction is necessary in order to convey the property to SFWMD.

The cleanup of the mill site involved assessment and remediation of a number of point source discharge areas. Areas of concern at the mill site included numerous leaking petroleum storage tanks, pesticide and/or arsenic impacted soils in the sediments of two drainage canals, an ash pit, a water storage retention area, and metals-impacted soils adjacent to several building slabs.

In general, the petroleum impacted areas were handled through excavation and on-site treatment of soils in ex-situ bioremediation piles. Once the treatment was verified by confirmation sampling, the treated soils were returned to their respective excavations. PSI excavated, treated, and backfilled approximately 16,000 cubic yards of petroleum impacted soil at the mill site.

PSI were instructed that the canals and surface depressions at the mill are to be filled as part of the EAA Reservoir A-1 construction. Therefore, rather than excavating impacted sediments from the drainage canals, PSI elected to cover these slightly impacted soils with a 2-foot cover. The cover is intended to prevent exposure of potential receptor species to these sediments. Pesticide and arsenic impacted soil was also excavated from other areas of concern at the mill site and consolidated in the ash pit. The ash pit was a low lying excavated area that accepted effluent from the boilers. The ash in the pit was lightly impacted with heavy metals and polynuclear aromatic hydrocarbons (PAHs). Additional soils from other areas of concern were also filled into the pit and the ash pit was covered with two feet of clean soil to prevent future exposure.

These three areas within the mill site where contaminated soils have been left and capped will also require restrictions on excavation activities. These parcels are identified as the South Rock Canal, the Ash Pit, and the Waste Lake Discharge Ditch. An additional area of capped, impacted soil is present approximately three miles west of the mill at the former borrow pit/agricultural landfill. These areas contain pesticide, PAH and metal impacted soils which are buried beneath a clean soil cover. The excavation restrictions are necessary to prevent disturbance of these areas. These areas have been surveyed by a professional land surveyor and the coordinates have been provided to SFWMD personnel to ensure that no disturbance of these areas occurs.

In summary, all of the physical assessment and remediation intended by SFWMD has been completed on all of the Exclusion Area parcels and all of the technical documents relating to the cleanup have been reviewed and accepted by FDEP. Remaining outstanding activities are to record the appropriate deed restrictions on a few of the parcels. Once these activities are completed, it is expected that the FDEP will issue SRCOs on the remaining parcels and all of the parcels can be conveyed to SFWMD.

The Talisman Exchange and the environmental remediation described in the preceding paragraphs occurred before Black & Veatch's involvement with the project. Black & Veatch has been instructed that the SFWMD has accepted the standard of protection offered by the remediation. No further investigations into contamination are intended at this time. The BODR does not address any of these risks, and Black & Veatch accepts no responsibility of existing conditions as directed by the SFWMD.

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SECTION 3

DESIGN REQUIREMENTS AND CRITERIA

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3. DESIGN REQUIREMENTS AND CRITERIA

3.1 PROJECT LIMITS AND SITE DATUM

The EAA Reservoir A-1 Project limits are bounded by U.S. 27 on the east, STA-3/4 on the south, the Holey Land WMA on the southwest, and farmland in the EAA on the northwest and north.

The horizontal datum for this report is North American Datum of 1983 (NAD83); and vertical datum is North American Vertical Datum of 1988 (NAVD88). Some other studies and designs use the National Geodetic Vertical Datum of 1929 (NGVD29) as a vertical datum. The relationship between them is NAVD88 = NGVD29 – 1.4 feet.

3.2 FUNCTIONAL AND OPERATIONAL

3.2.1 Inflow to EAA Reservoir A-1

EAA Reservoir A-1 inflows simulated by the WBM consist of flows from the NNRC, Miami Canal, seepage collection canals, and precipitation. A more detailed description of each inflow is provided in Section 6.2.4.

Based on the results of the WBM, the average yearly inflow into the EAA Reservoir A-1 from the NNRC over the POS is approximately 362,000 acre-feet, with a maximum value of 640,198 acre-feet in water year 1980 and a minimum value of 129,000 acre-feet in water year 1989.

Based on the results of the WBM, the average yearly inflow into the EAA Reservoir A-1 from the Miami Canal over the POS is approximately 372,000 acre-feet, with a maximum value of 838,000 acre-feet in water year 1970 and a minimum value of 40,166 acre-feet in water year 1982.

3.2.2 Outflow from EAA Reservoir A-1

EAA Reservoir A-1 outflows simulated by the WBM consist of losses from evaporation, seepage, environmental deliveries, agricultural deliveries, and excess volume outflows. A more detailed description of each outflow is provided in Sections 6.2.5 and 6.2.6.

Releases from the EAA Reservoir A-1 include environmental deliveries via STA-3/4 and agricultural deliveries for the NNRC/Hillsboro Canal basin. The ECP 2010 and ECP 2015 version 5.4.2 runs simulate the amount of flow required from the EAA Reservoir A-1 on a daily basis to supply the environmental deliveries and the agricultural deliveries, respectively.

Based on the ECP 2015 run, the average annual environmental delivery supplied by the EAA Reservoir A-1 via STA-3/4 is approximately 685,000 acre-feet, with a maximum of 1,487,000 acre-feet in water year 1983 and a minimum of 103,000 acre-feet in water year 1990. The current average annual inflow into STA-3/4 is approximately 656,000 acre-feet (Piccone, 2005). The total deliveries over the POS are approximately 24,000,000 acre-feet.

Based on the ECP 2010 run, the average annual agricultural delivery supplied by the EAA Reservoir A-1 is 84,000 acre-feet, with a maximum of 160,000 acre-feet in water year 1985 and a minimum of 19,000 acre-feet in water year 1970. The total deliveries over the POS are approximately 3,000,000 acre-feet.

The EAA Reservoir A-1 yield is defined as the sum of the environmental and agricultural deliveries supplied by the EAA Reservoir A-1. The average annual yield of the EAA Reservoir A-1 is approximately 769,000 acre-feet, with a maximum of 1,538,000 acre-feet in water year 1983 and a minimum of 207,000 acre-feet in water year 1971. The total yield over the POS is approximately 26,900,000 acre-feet.

3.2.3 Water Quality

The EAA Reservoir A-1 will be designed to the CERP Level 2 requirement, which states that the EAA Reservoir A-1 will not contribute to the degradation of water quality releases. Of primary interest is the fate of phosphorus entering the EAA Reservoir A-1 from the NNRC and Miami Canals. The Dynamic Model for Stormwater Treatment Areas (DMSTA) was selected to predict amounts of phosphorus in the EAA Reservoir A-1, leaving the EAA Reservoir A-1, and deposited in the bottom sediments.

DMSTA was developed by Dr. Bill Walker and Dr. Bob Kadlec (both are private consultants for the DOI) to support the design of the STAs. Compared with typical marsh treatment areas in the STAs, CERP storage reservoir designs have greater mean depths, greater variations in depth, and potentially, longer water residence times. These factors can be expected to have significant effects on vegetation communities, phosphorus dynamics, and model calibrations.

DMSTA Version 2 (DMSTA2), which was first released in June 2005, was enhanced to support its application to the deeper CERP storage reservoirs. The reservoir module of DMSTA2 was calibrated using existing water quality data from 11 Florida reservoirs.

Application of DMSTA2 to the EAA Reservoir A-1 involved:

(1) Importing to DMSTA2 1965-2000 time series of available average daily flows in the NNRC and Miami Canals from the ECP 2010 model run

(2) Importing to DMSTA2 EAA agricultural and environmental deliveries from the ECP 2010 model run

(3) Assigning phosphorus concentrations to each of the daily canal flows described in step (1). DMSTA2 provided continuous daily estimates of water and phosphorus mass balances in the EAA Reservoir A-1 over the 36-year simulation period.

Using the POS, between 1965 and 2000, the average total phosphorus loading to the EAA Reservoir A-1 would be 65.2×10^3 kilograms (kg) per year. Of that, 76 percent came from the canals, while only 3 percent and 21 percent were from rainfall and re-cycled seepage, respectively. DMSTA2 estimated that the EAA Reservoir A-1 would achieve an average 17 percent reduction in the phosphorus loading from the canals.

It was concluded that the EAA Reservoir A-1 will not negatively impact water quality in the EAA. Phosphorus contained in the Supply Canal could be removed in the EAA Reservoir A-1 as simulated by the DMSTA2 model.

Details of the water quality modeling of the EAA Reservoir A-1 are provided in Appendix 3-2. DMSTA2 model documentation is provided in Appendix 3-3.

3.3 SERVICE LIFE

According to USACE Engineering Manuals EM-1110-2-3104, EM-1110-2-3102, and Major Pumping Station Engineering Guidelines, the design life for the new northeast pump station and any modifications to G-370 and G-372 pump stations will be 50 years. With proper maintenance, this design life can be achieved by following the guidance in these documents.

The mechanical equipment will require rehabilitation or replacement over the design life. The engines and pumps will operate intermittently but will require regular maintenance. The engines may require at least one major overhaul during the design life while the pump materials will be designed to provide long service life. The architectural and structural design of the pump stations will include elements that will require minimum maintenance and repair over the design life.

The design elements for the structural; civil; mechanical; electrical; instrumentation and control; architectural; plumbing; and heating, ventilation, and air conditioning (HVAC) are described in more detail in Sections 11 through 17.

3.4 PROJECT WORK LIMITS

The EAA Reservoir A-1 Project limits are bounded by U.S. 27 on the east, STA-3/4 on the south, the Holey Land WMA on the southwest, and farmland in the EAA on the northwest and north. A survey of some of the cross-sections along the boundary of the EAA Reservoir A-1 was completed in 2004 (Wantman Group, 2004). Any need for additional surveying will be evaluated during the preliminary design. Final surveys for the EAA Reservoir A-1 components will be completed when the approved facility locations have been finalized.

3.5 UNITS

The units and system of measurement will be in English.

3.6 CODES AND STANDARDS

3.6.1 General

- CERP Guidance Memoranda
- SFWMD Design Criteria Memoranda
- SFWMD Standard Design Guidelines adopted August, 2005
- Acceler8 Design Criteria Memoranda (DCM)

3.6.2 Site Work Design Criteria

Codes and standards: design and specification of all work shall be accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- American Association of State Highway and Transportation Officials (AASHTO)
- American National Standards Institute, Inc. (ANSI)
- American Society for Testing and Materials (ASTM)

- Americans with Disabilities Act Accessibility Guidelines for Buildings and Facilities (ADAAG)
- Asphalt Institute (AI)
- Federal Highway Administration (FHWA)
- Florida Department of Transportation (FDOT)
- Manual on Uniform Traffic Control Devices (MUTCD)
- South Florida Water Management District (SFWMD)
- Uniform Federal Accessibility Standards (UFAS)
- United States Army Corps of Engineers (USACE)

3.6.3 Geotechnical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein. Recommended and recognized standards from other organizations shall be used where required and approved to serve as guidelines for the design, fabrication, and construction when not in conflict with the standards referenced herein.

- American Society for Testing and Materials (ASTM)
- Design Manual for Roller Compacted Concrete (RCC) Spillways and Overtopping Protection, Portland Cement Association, 2002
- Engineering Manual (EM) 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams
  - EM 1110-2-1901, Seepage Analysis and Control For Dams
  - EM 1110-2-1902, Slope Stability
  - EM 1110-2-2006, Engineering Design – Roller Compacted Concrete
- Florida Building Code, 2004 Edition
- Florida Department of Transportation (FDOT)
- South Florida Water Management District (SFWMD)
- United States Army Corp of Engineers (USACE)

3.6.4 Architectural Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Florida Accessibility Code - Latest Edition
- Florida Building Code - 2004 Edition
- Occupational Safety and Health Administration - 29 CFR

3.6.5 Structural Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Aluminum Design Manual "Specifications for Aluminum Structures," 2000
- American Concrete Institute (ACI)
  - ACI 318-02 "Building Code Requirements for Reinforced Concrete"
  - ACI 350-01/350R-01 "Code Requirements for Environmental Engineering Concrete Structures and Commentary"
  - ACI 350.4R-04 "Design Considerations for Environmental Engineering Concrete Structures"
  - ACI 530 "Building Code Requirements for Masonry Structures"
  - ACI 530.1 "Specification for Masonry Structures"
- American Institute of Steel Construction, Inc. (AISC): Manual of Steel Construction, Allowable Stress Design, 9th Edition
- American Society of Civil Engineers (ASCE) 7-02: Minimum Design Loads for Buildings and Structures
- American Welding Society (AWS)
  - American Welding Society, Structural Welding Code – Steel
  - American Welding Society, Structural Welding Code – Stainless Steel
  - American Welding Society, Structural Welding Code – Aluminum
- CERP Standard Design Manual, June 6, 2003
- Concrete Reinforcing Steel Institute Handbook
- Florida Building Code, 2004 Edition
- PCI Design Handbook, Precast and Prestressed Concrete
- South Florida Water Management District, Major Pumping Station Engineering Guidelines, November 29, 2004
- United States Army Corps of Engineers (USACE)
  - EM 1110-1-2009 Architectural Concrete
  - EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures, dated 1 February 1994
  - EM 1110-2-2102 Waterstops and Other Preformed Joint Materials for Civil Works Structures, dated 30 September 1995
  - EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures, dated 30 June 1992

- EM 1110-2-2105 Design of Hydraulic Steel Structures, dated 31 March 1993
- EM 1110-2-2502 Retaining and Flood Walls, dated 29 September 1989
- EM 1110-2-2701 Vertical Lift Gates, dated 30 November 1997
- EM 1110-2-3104 Structural and Architectural Design of Pumping Stations, dated 30 June 1989

3.6.6 Special Mechanical Equipment Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- American Association of State Highway and Transportation Officials (AASHTO)
- American Bearing Manufacturers Association (ABMA)
- American Gear Manufacturers Association (AGMA)
- American Petroleum Institute (API)
    - API Standard 620 - Design and Construction of Large Low Pressure Storage Tanks
    - API Standard 650 - Welded Steel Tanks for Oil Storage
- American Society of Mechanical Engineers (ASME)/ American National Standards Institute (ANSI)
    - ANSI/ASME B1.20.1 - General Purpose Pipe Threads
    - ANSI/ASME B16.1 - Cast Iron Pipe Flanges and Flanged Fittings, Class 25, 125, 250 and 800
    - ANSI/ASME B16.5 - Steel Pipe Flanges and Flanged Fittings
    - ANSI/ASME B16.11 - Forged Fittings, Socket-Welding and Threaded
    - ANSI/ASME B16.21 - Nonmetallic Flat Gaskets for Pipe Flanges
    - ANSI/ASME B16.25 - Butt-welding Ends
    - ANSI/ASME B31.10 - Pressure Piping
- American Society of Testing and Materials (ASTM)
    - ASTM A36 - Structural Steel
    - ASTM A53 - Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
    - ASTM A105 - Forgings, Carbon Steel for Piping Components
    - ASTM A139 - Electric Fusion Welded Steel Pipe
    - ASTM A139B - Specification for Electric-Fusion (Arc)-Welded Steel Pipe
    - ASTM A181 - Forgings, Carbon Steel for General Purpose Piping
    - ASTM A283 - Carbon Steel Plate, Shapes, or Bars

- ASTM A307 - Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile
- ASTM A312 - Specification for Seamless and Welded Austenitic Stainless Steel Pipe
- ASTM A563 - Specifications for Carbon and Alloy Steel Nuts
- ASTM A568 - Steel, Sheet, Carbon, and High Strength, Low Alloy Hot Rolled and Cold Rolled
- ASTM A570 - Hot Rolled Carbon Steel Sheet
- ASTM F593 – Stainless Steel Bolts, Hex Nuts, Screws, and Studs, 2000

- American Water Works Association (AWWA)
  - AWWA C200 - Steel Water Pipe 6 Inches and Larger
  - AWWA C207 - Steel Pipe Flanges for Waterworks Service, Sizes 4 Inch through 144 Inch
  - AWWA C208 - Dimensions for Fabricated Steel Water Pipe Fittings
  - AWWA M11 - Steel Water Pipe - A Guide for Design and Installation
  - AWWA C600 - Installation of Ductile-Iron Water Mains and their Appurtenances
- ANSI/ASME B36.10 - Welded and Seamless Wrought Steel Pipe
- CERP Standard Design Manual, 2003, USACE Jacksonville District and SFWMD
- EPA Regulation 40 CFR Part 280.41
- Heat Exchange Institute (HEI)
- Hydraulics Institute Standards (HI)
  - ANSI/HI Standard 9.8-1998 - Pump Intake Design
  - ANSI/HI Standard 2.1-2.6-2000 - Standards for Vertical Pumps
  - ANSI/HI Standard 9.6.1-1998 – NPSH Margin
- Manufacturers Standardization Society of Valve and Fitting Industry (MSS)
  - MSS-SP 58 (1993) Pipe Hangers and Supports + Materials, Design, and Manufacture
  - MSS-SP 69 (1996) Pipe Hangers and Supports + Selection and Application
- National Fire Protection Association (NFPA)
  - NFPA 30 - Flammable and Combustible Liquids Code
  - NFPA 30A - Automotive and Marine Station Code
  - NFPA 37 - Stationary Combustion Engines and Gas Turbines
  - NFPA 329 - System Test
- Pipe Fabrication Institute (PFI):
  - PFI-ES5 - Cleaning of Fabricated Pipe

- Steel Structures Painting Council (SSPC)
  - SSPC SP1 - Solvent Cleaning
  - SSPC SP3 - Power Tool Cleaning
  - SSPC SP5 - White Metal Blast Cleaning
  - SSPC-SP6 - Commercial Blast Cleaning
  - SSPC SP7 - Brush Off Blast Cleaning
- South Florida Water Management District, Major Pumping Station Engineering Guidelines, November 29, 2004,
- Underwriters Laboratories Inc. (UL)
  - UL-142 - Steel Aboveground Tanks for Flammable and Combustible Liquids
- United States Army Corps of Engineers (USACE)
  - EM 1110-2-3102, General Principles of Pumping Station Design and Layout, 1995
  - EM 1110-2-3104, Structural and Architectural Design of Pumping Stations, 1989
  - EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, 1999

3.6.7 HVAC, Plumbing and Fire Suppression

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. In addition to the applicable codes and standards previously identified, the system designs will also be based on but not limited to the following publications and standards:

- American Society of Heating, Refrigeration, and Air Conditioning Engineers (ASHRAE) Handbooks and Standards
- American Society of Plumbing Engineers (ASPE) Handbooks
- Florida Building Code 2001 – Mechanical
- Florida Building Code 2001 – Plumbing
- Florida Fire Protection Code
- Major Pumping Station Engineering Guidelines, 2004, South Florida Water Management District
- National Fire Protection Association Recommended Practices (NFPA) and Manuals
- Occupational Safety and Health Act (OSHA) Standards Manual
- Sheet Metal and Air Conditioning Contractor National Association (SMACNA) Handbooks

3.6.8 Fire Protection and Detection Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- International Building Code (International Code Council) - 2003
- International Fire Code (ICC) - 2003
- National Fire Protection Association (NFPA)
- Occupational Safety and Health Administration (OSHA)
- Underwriters' Laboratories, Inc. (UL)

3.6.9 Electrical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- American National Standards Institute (ANSI)
  - ANSI C2, National Electrical Safety Code
  - ANSI C84.1, Electric Power Systems and Equipment - Voltage Ratings
  - ANSI A117.1, Buildings and Facilities - Providing Accessibility and Usability for Physically Handicapped People
  - ANSI/IEEE Std. 242, Recommended Practice for Protection and Coordination of Industrial and Commercial Power Systems (The Buff book)
- Institute of Electrical and Electronics Engineers (IEEE) C62.41 Surge Voltage in Low Voltage AC Power Circuits
- Illuminating Engineering Society (IES) Lighting Handbook, Reference Volume and Application Volume
- National Fire Protection Agency (NFPA)
  - NFPA 70, National Electrical Code
  - NFPA 72, National Fire Alarm Code
  - NFPA 101, Code for Safety to Life from Fire in Buildings and Structures
  - NFPA 78, Lightning Protection Code
- Uniform Federal Accessibility Standards (UFAS)
- UL 268, Smoke Detectors for Fire Protective Signaling Systems
- USACE Technical Standards, TI-800-01

3.6.10 Instrumentation and Controls Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- American National Standards Institute (ANSI)
  - ANSI C37.90 (1989) Relays and Relay Systems Associated with Electric Power Apparatus
  - ANSI C37.90.1 (1989) Surge Withstand Capability (SWC) Test for Protective Relays and Relay Systems
  - EM ANSI/EIA/TIA -232-F (2002) Interface Between Data Terminal Equipment and Data Circuit-Terminating Equipment Employing Serial Binary Data Interchange
- Institute of Electrical and Electronics Engineers (IEEE)
  - IEEE C62.41 (1991) Recommended Practice for Surge Voltages in Low-Voltage AC Power Circuits
  - IEEE Std 100 (2000) IEEE Standard Dictionary of Electrical and Electronics Terms
  - IEEE Std 802 (1990; R 1995) Information Processing Systems, Local Area Networks: Part 4: Token Passing Bus Access Method and Physical Layer Specifications
- International Electrotechnical Commission (IEC) 61131-3 (2003) Programmable Controllers — Part 3: Programming Languages
- National Electrical Manufacturer's Association (NEMA)
  - NEMA 250 (1997) Enclosures for Electrical Equipment (1,000 Volts Maximum)
  - NEMA ICS 1 (2000) Industrial Control and Systems: General Requirements
  - NEMA ICS 2 (2000) Industrial Control and Systems: Controllers, Contactors, and Overload Relays Rated 600 volts
  - NEMA ICS 4 (2000) Industrial Control and Systems: Terminal Blocks
  - NEMA ICS 6 (1993; R 2001) Industrial Control and Systems: Enclosures
- National Fire Protection Agency (NFPA) 70 (2002) National Electrical Code
- Underwriter's Laboratories
  - UL 1059 (2001) Terminal Blocks
  - UL 508 (1999; Rev thru Dec 2002) Control Equipment

3.6.11 Telemetry System Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the State of Florida and the federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Electronics Industries Alliance (EIA)
  - EIA ANSI/TIA/EIA-222-F (1996) Structural Standards for Steel Antenna Towers and Antenna Supporting Structures
  - EIA ANSI/EIA/TIA-232-F (2002) Interface Between Data Terminal Equipment and Data Circuit Terminating Equipment Employing Serial Binary Data Interchange
  - EIA ANSI/EIA-310-D (1992) Racks, Panels, and Associated Equipment
- Federal Communications Commission (FCC) 47 CFR 15 Radio Frequency Devices
- SFWMD Design Standards and Guidelines

3.6.12 Design Criteria Memoranda

Following is a summary of the Design Criteria Memoranda and their respective issue dates.

- DCM-1 Hazard Potential Classification — August 19, 2005
- DCM-2 Wind and Precipitation Design Criteria for Freeboard — October 11, 2005
- DCM-3 Spillway Capacity and Reservoir Drawdown Criteria — August 19, 2005
- DCM-4 Minimum Dimensions of Dams and Embankments — August 19, 2005
- DCM-5 Major Pump Station Engineering Guidelines — In Progress
- DCM-6 Geotechnical Seismic Evaluation of CERP Dam Foundations — May 16, 2005
- DCM-7 Procedure for Development of Engineering Construction Cost Estimates — August 5, 2005
- DCM-8 Vulnerability Protection Requirements — In Progress
- DCM-9 Embankment Instrumentation — In Progress
- DCM-10 Construction Quality Assurance Procedures — In Progress
- DCM-11 Post Construction/Inspection/Dam Safety Program — In Progress
- DCM-12 Value Engineering — In Progress

3.7 *REFERENCES*

Copp, R. (A.D.A. Engineering, Inc.), Personal Communication, July 12, 2005.

Office of Modeling, *ECP 2010 and 2015 Model Runs (Version 5.4.2).* South Florida Water Management District, June, 2005.

Piccone, T. Personal Communication. E-mail dated July 19, 2005.

Wantman Group. *South Florida Water Management District Storage Reservoir Specific Purpose Survey*, May, 2004.

SECTION 4

REGULATORY CONSIDERATIONS AND ASSURANCES

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TABLE

4. REGULATORY CONSIDERATIONS AND ASSURANCES

4.1 INTRODUCTION

This Section summarizes the regulatory and permitting requirements within the State of Florida that may be required for the EAA Reservoir A-1 Project. The permits necessary to construct and operate the EAA Reservoir A-1 will depend on the results of agency consultation and/or field surveys as applicable as well as the final construction design. Table 4.1-1 (located at the back of this Section) summarizes the federal and state permitting requirements and provides an approximate timeline for obtaining each permit. The following is a discussion of applicable permits and associated guidance that has been assembled for this EAA Reservoir A-1 Project.

4.2 FEDERAL REQUIREMENTS

The EAA Reservoir A-1 Project is required to satisfy many federal requirements including the NEPA, Clean Water Act (CWA), Clean Air Act (CAA), Endangered Species Act (ESA), Rivers and Harbors Act, Coastal Zone Management Act (CZMA), Fish and Wildlife Coordination Act of 1958, National Historic Preservation Act, Safe Drinking Water Act (SDWA), Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), Title V, Section 106 Coastal Resources Marine Mammal Protection Act, Migratory Bird Treaty Act, Migratory Bird Conservation Act, Resource Conservation and Recovery Act (RCRA), and WRDA Sections 904, 307, and 601, while keeping in mind such issues as, protection of wetlands, floodplain management, environmental justice, and invasive species.

4.2.1 National Environmental Policy Act (NEPA) Coordination

NEPA requires that federal agencies consider the potential effects of actions that may adversely affect the environment and consider possible alternative courses of action to reduce impacts before approving the EAA Reservoir A-1 Project. As a federally funded project, the EAA Reservoir A-1 Project will require a NEPA environmental analysis. Due to the nature and size of the EAA Reservoir A-1 Project, an EIS is required. The USACE is in the process of preparing the EIS for this EAA Reservoir A-1 Project as part of the PIR process (USGS 2004).

4.2.2 USACE Dredge & Fill Permit

Section 404 of the CWA and the Rivers and Harbors Act (Section 10) govern the disposal of dredged material or fill in the nation's waters, including wetlands. The USACE is charged with overseeing the regulation of dredging and filling activities in waters of the United States. Persons wanting to dispose of dredged material or conduct infill activities in waters of the U.S. are required by law to obtain a permit from the USACE. The EAA Reservoir A-1 Project will involve disposal of dredged material or fill activities within waters of the U.S., specifically jurisdictional project wetlands.

The USACE requires either Section 404 or Section 10 permits to fill in wetlands and surface waters of the U.S. The types of Dredge and Fill permits applicable in Florida are described in Sections 2.2.1 through 2.1.5 of Section 9 of the Rivers and Harbors Act of 1899 (USACE 2004b). A Section 10 permit is not expected to be applicable to the proposed EAA Reservoir A-1 Project, since none of the waters involved is designated as navigable. Key commenting agencies for the USACE permit include the EPA, USFWS, and State Historic Preservation

Office (SHPO). The EPA is authorized to prohibit the use of a site for disposal if discharges would have an unacceptable, adverse effect on municipal water supplies, shellfish beds, fishery areas, wildlife, or recreational uses. Some types of activities are exempt from permit requirements, including certain farming, ranching, and forestry practices, which do not alter the use or character of the land; some construction and maintenance; and activities already regulated by states under other provisions of the CWA (United States Coast Guard [USCG] 2002).

An Individual Permit application is required for projects that cannot meet the minimal impact requirements of Nationwide Permits or are not covered by available Regional General Permits. The EAA Reservoir A-1 Project would require an Individual Permit under the CWA Section 404. Permit application requirements include: detailed site information, project details, site plans, existing and proposed environmental conditions, construction, drainage, and operational information.

The permit application requires a 30-day public comment period. During this time, interested parties and local, state, and federal agencies are allowed to review and comment on the proposed EAA Reservoir A-1 Project (USACE, 2004a). After all comments have been evaluated a final decision from the USACE will be made based on internal comments and those from the public. The USACE will release a Statement of Findings explaining the permit decision and issue the permit (EPA, 2005).

4.2.3 Section 401 Water Quality Certification

Section 401 of the CWA requires state water quality certification prior to federal sponsorship or issuance of federal permits, such as the USACE Section 404 Dredge and Fill Permit. F.S. Section 373.1502(3)(b)(2) states that, “state water quality standards will be met to the maximum extent practicable. Under no circumstances shall the project component cause or contribute to violations of state water quality standards.” Water quality certification is provided by the State of Florida. According to Section 373.1502 F.S., since the EAA Reservoir A-1 Project is part of the CERP, water quality certification will be issued by the state concurrently with the Comprehensive Everglades Restoration Plan Restoration Act (CERPRA) Permit.

4.2.4 SPCC Plans – CWA

Under the provisions of EPA 40 CFR Part 112 - Oil Pollution Prevention, facilities which have above ground petroleum products storage of greater than 1,320 gallons aggregate or greater than 42,000 gallons below ground storage, are required to have a Spill Prevention, Control and Counter Measures (SPCC) Plan which meets all of the requirements of this regulation. Part 112 of 40 CFR establishes procedures, methods and equipment, and other requirements to prevent the discharge of oil from facilities into or upon the navigable waters of the United States or adjoining shorelines. The SPCC Plan must be formulated under the supervision of and certified by a registered professional engineer and must be available at the facility for inspection by FDEP and EPA personnel. If above ground petroleum products storage exceeds the above thresholds, a SPCC Plan will be required.

4.2.5 National Pollution Discharge Elimination System Permits (NPDES)

The National Pollution Discharge Elimination System (NPDES) regulations for discharges of industrial effluent are authorized under the provisions of Section 402 of the CWA. Authority for administration of the NPDES Program is delegated to the FDEP by the EPA. Please refer to State Permitting Requirements below for details of the State implementation of this program.

4.2.6 Coastal Zone Management

The CZMA authorizes the National Oceanic and Atmospheric Administration (NOAA) to administer the federal Coastal Zone Management Program (CZMP). The purpose of CZMP is to preserve, protect, develop, and where possible, to restore or enhance, the resources of the nation’s coastal zone. The federal CZMP has delegated the day-to-day management of the program in Florida to the Florida Department of Community Affairs, including determinations of consistency (FDEP 2005b).

The CZMP coordinates state governmental activities related to the protection, preservation, and development of Florida's natural, cultural, and economic coastal resources. A network of 10 agencies implement the program (FDEP 2005b). Federal projects, such as the EAA Reservoir A-1 Project, are typically reviewed for consistency on an individual basis during the PIR/EIS process. Because this is a federally mandated program, the EAA Reservoir A-1 Project will be required to obtain Coastal Zone Consistency from the State of Florida.

4.2.7 Section 7 Endangered Species Act Consultation

Section 7 of the ESA requires that federal actions not jeopardize the continued existence of federally listed species nor modify their designated critical habitat. Based on initial consultations with the USFWS, protected species may inhabit the EAA Reservoir A-1 Project area. Several studies have been and/or are in progress. Informal consultation with the USFWS should be conducted to develop an EAA Reservoir A-1 Project plan that will not harm federally listed species. Refer to USFWS Environmental Existing Conditions report for potential protected species observed within the EAA Reservoir A-1 site. Those threatened, endangered, or protected species that may be found on the EAA Reservoir A-1 site are listed in Appendix 4-1.

4.2.8 Section 106 - Cultural Resources

The National Historic Preservation Act (NHPA) (16 U.S.C. 470) created the Advisory Council on Historic Preservation (ACHP), an independent federal agency, to advise the President and Congress on matters involving historic preservation. The ACHP is authorized to review and comment on all actions licensed by the federal government which will have an effect on properties listed in the National Register of Historic Places, or eligible for such listing.

Section 106 (16 U.S.C. 470f) requires that any federal agency having direct or indirect jurisdiction over a proposed federal or federally assisted project review that project for impact on significant historic properties. The agency must allow the SHPO and the ACHP to comment on a proposed project to determine whether cultural/historic resources can be found on the project site. If significant potential exists for discovering cultural resources, SHPO may request additional studies for clarification. If historic or cultural resources are found, the agency must determine the effects on those properties and seek ways to avoid or reduce any negative effects.

Where excavation is proposed for the EAA Reservoir A-1, a cultural resources survey and confirmation of compliance with Section 106 and the NHPA will be required.

4.2.9 Clean Air Act

The northeast pump station proposed for the EAA Reservoir A-1 may affect air quality; thus compliance with this act will be required. However, under the current delegation agreement between the EPA and FDEP, permit review is administered at the state level and reviewed for concurrence with CAA Requirements by the EPA. Please refer to State Permitting Requirements below for further details.

4.2.10 Miscellaneous

USFWS (Slack, 2005) issued a Planning Aid Letter (PAL) to USACE on March 11, 2005, in which they provided guidance and recommendations on resource conservation issues for the EAA Reservoir A-1 Project. USFWS recommended including a habitat buffer on the north and west sides of the EAA Reservoir A-1 and littoral shelves along the seepage canals and on the internal sides of the embankment. USFWS recognized that littoral shelves on the interior sides of the EAA Reservoir A-1 may be cost-prohibitive.

4.3 PROJECT ASSURANCES

Within WRDA 2000, Congress approved CERP’s objectives to restore, preserve, and protect the south Florida ecosystem while providing for water-related needs of the region. The components of CERP will increase storage and water supply for the natural system, as well as for agricultural and urban needs. Provisions in Section 601(h) of the WRDA, “Assurance of Project Benefits,” resulted in an agreement between the federal government and the State of Florida. Because implementation of CERP will require the cooperation and collaboration of federal, state, and tribal entities, all interests sought assurances that they would receive the anticipated benefits from CERP.

The Federal-State Agreement states the following,

> “As required by the Water Resources Development Act of 2000, water made available by each project in the Comprehensive Everglades Restoration Plan will not be permitted for a consumptive use or otherwise made unavailable by the State of Florida until such time as sufficient reservations of water for restoration of the natural system are made by regulation or other appropriate means pursuant to Section 373, Florida Statutes, and in accordance with the project implementation report for the project and consistent with the Comprehensive Everglades Restoration Plan.”

Section 601(h)(4) of the WRDA specifies that a PIR will be used to document consistency with CERP; to satisfy the programmatic regulations; to comply with the NEPA; to identify the appropriate quantity, timing, and distribution of water dedicated and managed for the natural system; to comply with water quality standards and permitting requirements; to identify the amount of water to be reserved or allocated for the natural system necessary to accomplish the quantity and quality objectives; to be based on the best available science, and to include an analysis of cost-effectiveness and engineering feasibility of the EAA Reservoir A-1 Project.

Section 601(h)(5) of the WRDA specifies a savings clause that must be considered when implementing a project under CERP. Protection of existing legal sources from elimination or transfer and protection of level of service of flood protection existing as of December 2000 is required by the federal law. The PIR will contain the analyses required to determine whether an elimination or transfer has occurred as a result of implementation of CERP and whether levels of service for flood protection will be reduced. These analyses will be conducted on the alternative plan selected by the SFWMD from the BODR.

Implementation of CERP must also be consistent with State law. As a local sponsor, SFWMD has responsibilities that are outlined in Section 373.1501(5) of the F.S. Subsection (d) requires the SFWMD to do the following:

> "… provide reasonable assurances that the quantity of water available to existing legal users shall not be diminished by implementation of project components so as to adversely impact existing legal users, that existing levels of service for flood protection will not be diminished outside the geographic area of the project component, and that water management practices will continue to adapt to meet the needs of the restored natural environment."

After the preferred alternative EAA Reservoir A-1 Project plan is selected by the SFWMD and the spatial extent of the EAA Reservoir A-1 effects is identified, separate comparisons of modeling simulations will be performed to satisfy the federal (WRDA 2000) and state (Section 373.1501 F.S. assurances), and to identify the water made available for the protection of fish and wildlife and for other water related needs. Separate comparative analyses are planned by the SFWMD to evaluate the following conditions:

- Section 373.1501 F.S. - Assurances analysis to evaluate the quantity of water available to existing legal users
- Section 373.1501 F.S. – Assurances analysis to evaluate the effects of EAA Reservoir A-1 Project implementation on existing levels of flood protection
- WRDA 2000 - Quantification of water made available by the EAA Reservoir A-1 Project for the protection of fish and wildlife
- WRDA 2000 - Quantification of water made available by the EAA Reservoir A-1 Project for other water related needs

To evaluate the potential effects of the EAA Reservoir A-1 Project on existing levels of flood protection, the "Existing PIR Baseline" condition will be compared to the Existing PIR Baseline with the EAA Reservoir A-1 in place to determine whether any significant and adverse impacts result from the EAA Reservoir A-1 Project. To accomplish this, sub-regional modeling will be conducted by the SFWMD combined with the MODFLOW (a three-dimensional groundwater flow model) and SEEP/W (a two-dimensional finite element model) models to determine the EAA Reservoir A-1 Project's potential effects on the level of service for flood protection. It is planned that this series of model runs will be performed following the preparation of the BODR and prior to completion of the 30-Percent Engineering Design Report.

The SFWMM 2x2 model (which is the same as the ECP 2010 and 2015, Version 5.4.2, runs for the Regional Feasibility Study) is the primary tool used by the SFWMD to evaluate the interaction of water supply and water deliveries with hydrologic conditions on a regional scale. Because the regional model is based on 2-mile square grid cells, a sub-regional model with greater detail at a local scale will be developed by SFWMD to simulate the operation of the EAA Reservoir A-1. The localized, sub-regional model will be used by SFWMD to quantify the water made available by the EAA Reservoir A-1 Project for the protection of fish and wildlife and for other water related needs, as well as to evaluate any elimination or transfer of water that may result from the EAA Reservoir A-1 Project. As with the aforementioned subregional modeling, it is planned that the SFWMD will perform this series of model runs following preparation of the BODR and prior to completion of the 30-Percent Engineering Design Report.

Project assurance had been considered in a preliminary manner as part of the evaluation of engineering alternatives for the EAA Reservoir A-1 BODR. The assurances are addressed in other sections of the BODR:

- Alternatives to control seepage from the EAA Reservoir A-1 and provide protection from flooding are described in Section 9.
- Modeling results, which describe the environmental deliveries to the WCAs, are presented in Section 6.
- Modeling results, which describe the agricultural deliveries to the farm lands, are presented in Section 6.
- The EAA Reservoir A-1 will be operated to store water, which would otherwise be lost to tide or sent to the WCAs during wet seasons. The stored water can now be made available for environmental or agricultural deliveries at more appropriate timing. This is presented in the Operations Plan described in Section 20.

4.4 STATE PERMITS

The FDEP is responsible for reviewing the majority of environmental permits. Section 373.1502(3)(b) F.S. authorizes the FDEP to issue permits for the construction, operation and maintenance of CERP project components under the CERPRA unless either of the following conditions applies: (1) the project component is otherwise subject to the EFA or the Lake Okeechobee Protection Act (LOPA), or (2) the project is subject to the FDEP's rules on reuse of reclaimed water. The EAA Reservoir A-1 Project is not included as part of the EFA or LOPA, and is therefore, subject to the regulations set forth in CERPRA.

The FDEP is responsible for administering the CZMP and the use of Sovereign State Lands. Descriptions of the permits and clearances, which may be needed for the EAA Reservoir A-1 Project, are provided in the following sections.

4.4.1 Comprehensive Everglades Restoration Plan Act Permit

The EAA Reservoir A-1 Project is not included as part of the EFA or LOPA, and is therefore, considered part of the CERPRA. CERPRA projects are subject to Section 373.1502 F.S. With the exception of federally delegated or approved permitting programs, permits issued pursuant to Section 373.1502 F.S. are in lieu of all other permits and authorization required under Chapter 373 F.S., Chapter 403 F.A.C. Therefore, most state permits will be issued concurrent with the CERPRA permit.

The FDEP will issue a CERPRA permit for a term of five years providing the applicant can provide reasonable assurances that:

- The project component will achieve the design objectives set forth in the detailed design documents submitted as part of the application
- State water quality standards, including water quality criteria and moderating provisions, will be met. Under no circumstances shall the project component cause or contribute to violation of state water quality standards
- Discharges from the project component will not pose a serious danger to public health, safety, or welfare
- Any impacts to wetlands or threatened or endangered species resulting from implementation of the project component will be avoided, minimized, and mitigated, as appropriate

A comprehensive plan amendment is not required based on discussions with representatives of Palm Beach County.

4.4.2 Well Construction Permit

A permit is required for the construction, repair, or abandonment of any well in the SFWMD unless specifically exempted by rule or law. The construction and/or repair of water wells, including monitoring wells, must be performed by a Florida-licensed water well contractor. Chapter 62-532 F.A.C. of the FDEP Rules regulates all such activities under the Florida Department of Health (DOH, 2005). SFWMD is responsible for issuing Well Construction Permits in Palm Beach County for wells four inches or greater in diameter. The Palm Beach County Public Health Unit (CPHU) is delegated the authority for issuance of permits for wells less than four inches in diameter. The alternatives, which use wells to control seepage, are described in Section 9.3.4 and 9.3.5 and propose 6 inch diameter wells.

4.4.3 Consumptive Use Permit

Water management districts regulate the Consumptive Use Permit program in Florida, as prescribed in Chapter 373, Part II F.S. Two types of permits may be issued: Individual and General. If a specified project will exceed a monthly use of 15 million gallons per month (MGM) an individual permit is required. A minor General Permit (GP) will be issued if water consumptions estimated to be less than 3 MGM; a major GP is issued when use is estimated to be between 3 MGM and 15 MGM. To satisfy the permit requirements, the applicant must be able to show that the use will be "reasonable and beneficial". Secondly, the applicant must demonstrate that the use is consistent with the public interest. The third provision of the permit evaluation requires the applicant to assure that the use will not result in adverse impacts to existing legal users (SFWMD, 2005). Because the EAA Reservoir A-1 Project will be permitted under CERPRA, a Consumptive Use Permit will not be required in accordance with Section 373.323 F.S. During the CERPRA process, however, a consumptive use evaluation will be conducted. All pertinent information, including the requirements listed above should be submitted with the CERPRA application.

4.4.4 Florida Department of Transportation Access Permit

An Access Permit is required from the FDOT for driveways, streets, turnouts, or other means of providing access to the state highway system. Rule 14-96 and 14-97 F.A.C. govern access permits. Any access road constructed for the EAA Reservoir A-1 Project, which will connect to the state highway system, will require an access permit from the FDOT. A GP will be required for the construction of the planned bridge on U.S. 27, coordination with FDOT should occur as early into the EAA Reservoir A-1 Project planning as possible to prevent schedule delays. In addition, the EAA Reservoir A-1 Project would be required to undergo Maintenance of Traffic review to determine any necessary traffic improvements resulting from the EAA Reservoir A-1 Project.

4.4.5 Clean Air Construction (NSR-PSD) and Operation (Title V) Permits

Under the terms of its delegation of CAA permitting authority from the EPA, the FDEP is responsible for New Source Review (NSR) of proposed stationary sources of air pollution and issuance of both Construction (Prevention of Significant Deterioration or PSD) and Operation (Title V) Permits, as applicable. Following is a discussion of the permitting program as it may apply to the EAA Reservoir A-1 Project.

4.4.5.1 New Source Review (NSR) and Prevention of Significant Deterioration (PSD) Construction Permits

It is possible that a Categorical Exemption (CE) could apply to the EAA Reservoir A-1 pump station if the facility uses less than certain amounts of fuel on an annual basis. A CE is allowed for the following:

> "…one or more heating units and general purpose internal combustion engines located within a single facility provided none of the heating units or general purpose internal combustion engines is subject to the Federal Acid Rain Program, and total fuel consumption by all such heating units and general purpose internal combustion engines within the facility is limited to 32,000 gallons per year of diesel fuel, 4,000 gallons per year of gasoline, 4.4 million cubic feet per year of natural gas or propane, or an equivalent prorated amount if multiple fuels are used (Section 62-210.300(3)(a)21 F.A.C.)."

However, it is expected that the northeast pump station will require more than 32,000 gallons of diesel fuel for operation each year.

In addition, a Generic Exemption may be available based on performance parameters is provided in Section 62-210.300(3)(c)3 F.A.C. Facilities comprising heating units and general-purpose internal combustion engines are exempt, provided the following conditions are met:

- The facility operates no emissions units other than the heating units and general-purpose internal combustion engines.
- None of the heating units or general purpose internal combustion engines is subject to the Federal Acid Rain Program as defined at Rule 62-210.200, F.A.C.
- Each of the heating units or general purpose internal combustion engines meets the general visible emissions standard of Rule 62-296.320(4)(b), F.A.C.
- Total fuel consumption by all heating units and general purpose internal combustion engines within the facility is limited to 250,000 gallons per year of diesel fuel, 30,000 gallons per year of gasoline, 35 million cubic feet per year of natural gas or propane, or an equivalent prorated amount if multiple fuels are used.

- The owner or operator of the facility maintains records to document the fuel consumption, by type, for each emissions unit. The owner or operator shall retain these records, available for FDEP inspection, for a period of at least five years.
- The owner or operator submits a completed Heating Units and General Purpose 62-210-40 Internal Combustion Engines Air General Permit Notification Form, showing entitlement to the use of the general permit, to the FDEP at least 30 days prior to beginning operation.

The final determination as to the level of involvement which the EAA Reservoir A-1 Project will have with respect to Stationary Source Permitting cannot be made until pump station design, and the pump selection process have commenced.

If the above exemption criteria are not met, then the EAA Reservoir A-1 Project must demonstrate compliance with the general PSD Requirements of the CAA. Emissions estimates would need to be developed for the CAA-designated Criteria Air Pollutants, which are carbon monoxide, lead, nitrogen dioxide, ozone, particulate matter, and sulfur dioxide. Additionally, National Ambient Air Quality Standards (NAAQS) have been established for these pollutants. If required to comply with PSD, the EAA Reservoir A-1 Project would need to demonstrate by air dispersion modeling that pump operations would not cause exceedances of these standards at the EAA Reservoir A-1 Project property line.

4.4.5.2 Clean Air Act Operating (Title V) Permits

The FDEP is responsible for air operating permits, which regulate both major and minor emitters. Operating permits are legally enforceable documents that are issued to air pollution sources. As stated in Section 62-210.300 F.A.C., any emissions unit which emits or can reasonably be expected to emit any air pollutant needs to obtain an appropriate air permit from the FDEP prior to beginning construction, modification, or initial or continued operation of the emissions unit unless exempted pursuant to FDEP rule or statute (FDEP, 2004a). FDEP issues the following types of air permits:

- Title V Operating Permits are for sources of air pollution regulated by Title V of the CAA. These sources include those that are subject to acid rain rules, and those certified under the Power Plant Sighting Act.
- Title V General Permits are for area sources of air pollution, such as perchloroethylene (dry cleaners), chromium (electroplating and anodizing facilities), halogenated solvent degreasers, ethylene oxide sterilizers, asbestos manufacturers and fabricators, and secondary aluminum sweat furnaces.
- Non-Title V General Permits are for minor sources of air pollution, such as mercury recovery and reclamation, bulk gasoline plants, heating units and general purpose internal combustion engines, surface coating operations, plastic products fabrication, cast polymer operations, concrete batching plants, human crematory, animal crematory, and nonmetallic mineral processing plants.

The proposed EAA Reservoir A-1 Project could potentially require a Non-Title V General Permit for minor stationary sources of air pollutants, depending on the type of facilities required to operate and maintain the proposed EAA Reservoir A-1 and canal system. Section 62-210.300

F.A.C. establishes the rules and regulations governing Stationary Source Permits. According to this chapter, two pertinent exemptions may apply to the proposed EAA Reservoir A-1 Project. They are defined in Section 62-210.300(3) F.A.C. and relate to:

- Categorical exemptions related to fossil fuel steam generators and hot water generators
- Generic and Temporary Emissions Unit Exemptions are those not entitled to a Categorical Exemption that relate to emissions quantities.

4.4.6 Petroleum Storage Tanks

Petroleum storage tanks are regulated under the provisions of Rule 62-761 F.A.C. Underground Storage Tanks (USTs) and Rule 62-762 F.A.C. Aboveground Storage Tanks (ASTs). If the EAA Reservoir A-1 Project requires either UST storage capacity in excess of 110 gallons or AST storage capacity in excess of 550 gallons, then the Project must comply with the applicable standards for engineering, construction and operation of the storage system. Additionally, all regulated storage tanks must be registered with the FDEP and will be subject to annual inspections by the Palm Beach County Department of Environmental Resource Management.

4.4.7 Dewatering Permits

Dewatering permits are administered by the SFWMD under the provisions of Rule 40E-20 F.A.C for water use permits. A Dewatering Notice General Use Permit may be required for the EAA Reservoir A-1 Project if the dewatering total quantity is less than 10 million gallons per day (mgd), less than 1,800 MGD total pumpage and less than one year in duration. For dewatering quantities less than five MGD, 100 million gallons total and less than 90-days duration, no notice is required. However, all dewatering projects must meet the conditions of Section 40E-20.301 F.A.C., which prohibit adverse impacts to environmental resources and off-site existing legal water users.

4.4.8 National Pollution Discharge Elimination System (NPDES) Permits

The NPDES regulations for discharges are administered by the FDEP. The NPDES Generic Permit for Stormwater Discharge (GCP) permitting program is administered in accordance with Rule 62-621, F.A.C. and was authorized by Section 403.0885 F.A.C. Construction activities associated with the EAA Reservoir A-1 Project will likely be permitted under the FDEP’s Generic Permit for Stormwater Discharge from Large and Small Construction Activities (GCP) pursuant to Section 62-21.300(4)(a) F.A.C. This NPDES permit requires development of a Stormwater Pollution Prevention Plan (SWPPP). The SWPPP identifies potential sources of pollution that may affect the quality of storm water from the EAA Reservoir A-1 Project area and outlines methods to reduce sediment runoff that may affect storm water quality (National Environmental Technical Memorandum [NETM] 2005). In addition to the GCP, if any off-site discharges will occur due to construction dewatering activities, coverage under the NPDES General Permit for the Discharge of Produced Groundwater from any non-contaminated site activity may be required pursuant to Section 62-621.300(2) F.A.C. Before discharge of produced groundwater can occur, analytical tests on samples of the proposed discharge water shall be performed to determine if contamination exists. Results from analytical tests must be compared to the applicable criteria as identified in the GCP.

4.4.9 Dam Safety Permit

The FDEP is responsible for the State of Florida Dam Safety Program; the water management districts within the State are also authorized to regulate dams. The permitting process for construction of dams within the State of Florida is found within Chapter 373 F.S. Additionally, the Federal Emergency Management Agency (FEMA) has published a report titled Federal Guidelines for Dam Safety (April 2004). The height and size of the proposed embankment around the EAA Reservoir A-1 will determine which provisions apply.

4.5 LOCAL PERMITS

Local permitting authority for the EAA Reservoir A-1 Project resides with several Palm Beach County Departments and Divisions. Primary coordination of local permit review will be administered by the County's Planning, Zoning and Building (PZB) Division. Following is a list of County Departments and Divisions which will be involved in review of the EAA Reservoir A-1 Project.

4.5.1 Palm Beach County Planning, Zoning & Building (PZB) Division – Development Review

Under the Palm Beach County's Development Review Procedures, the EAA Reservoir A-1 Project may be required to obtain zoning approval and a building permit prior to construction. However, in discussions with County PZB Division staff, it was indicated that the County will first review the EAA Reservoir A-1 Project to determine if it may be exempted from this process under the provisions of the EFA, Section 373.4592 F.S. If not, the following reviews will be conducted.

4.5.1.1 Palm Beach County Fire Rescue

As a component of building code compliance review, the EAA Reservoir A-1 Project will be required to demonstrate compliance with County and NFPA Fire Codes regarding fire protection facilities and emergency response capabilities.

4.5.1.2 Palm Beach County Health Department

The CPHU will be responsible for permitting any potable water or domestic waste facilities both during the construction and post-construction phases of the EAA Reservoir A-1 Project. If temporary sanitary facilities or holding tanks are required to temporarily support construction personnel, these facilities will need to be permitted by the Palm Beach County Public Health Unit (CPHU). If permanent potable water and domestic waste facilities are planned, these facilities will require both construction and operation permits.

4.5.2 Palm Beach County Department of Environmental Resources Management

4.5.2.1 Vegetation Preservation and Protection

Vegetation removal activities in Palm Beach County are regulated under Article 14C of the Unified Land Development Code (ULD). In general, this ordinance requires a standard permit for vegetation removal on non-residential projects of any size. However, a "de minimis approval" may be available for projects involving removal of only invasive and nuisance species.

Compliance review will be conducted by the Palm Beach County Department of Environmental Resource Management (ERM).

4.5.2.2 Stormwater Pollution Prevention Permit - Ordinance No. 2004-050

Any project involving land disturbances greater than one acre in extent is required to comply with the provisions of this ordinance. These include the same general provisions as those of the State Generic Permit for Large and Small Construction Projects, including the requirement for SWPPP. In fact, a determination of compliance with this ordinance may be issued if a project is demonstrated to be in compliance with all applicable stormwater management regulations of the SFWMD and FDEP. Compliance review will be conducted by the Palm Beach County ERM.

4.5.2.3 Wellfield Protection - Article 14, Part B

The provisions of the Palm Beach County Wellfield Protection Ordinance are designed to prevent the contamination of the County's groundwater resources by regulated hazardous substances. The EAA Reservoir A-1 Project will be required to demonstrate that the storage, use and handling of hazardous substances during the EAA Reservoir A-1 Project will not have the potential to cause contamination of County drinking water resources. During County review, documentation of proper storage and containment facilities for hazardous chemicals to be used on the EAA Reservoir A-1 Project must be furnished and emergency response plans developed for the release of hazardous chemicals. Compliance review will be conducted by the County ERM.

4.5.2.4 Petroleum Storage Tanks – Ordinance No. 2003-020

The above ordinance provides authority for local enforcement of Rules 62-761 and 62-762 F.A.C., for regulation of USTs and ASTs, respectively. Under these state regulations, USTs with greater than 110 gallons capacity and ASTs with greater than 550-gallon capacity are required to follow all applicable engineering and performance standards for these systems. Under the terms of the delegation agreement between the FDEP and Palm Beach County, the County will review all plans for each regulated storage system and oversee the installation. Compliance review will be conducted by the County ERM.

4.6 REFERENCES

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Table 4.1-1 EAA Reservoir A-1 Project Federal and State Permitting Requirements

Permit/Approval	Regulated Activity	Agency and Contact	Authority	Approval Timeline*
FEDERAL PERMITTING REQUIREMENTS				
NEPA	Major Federal Action Affecting the Environment	U.S. Army Corps of Engineers Ms. Tori White South Permits Section Office, SESAJ-RD-SS 4400 PGA Boulevard, Ste. 500 Palm Beach Gardens, FL 33410-2933 561-472-3517	40 CFR Parts 1508	12-18 months
Section 404	Fill of wetlands	U.S. Army Corps of Engineers Ms. Tori White South Permits Section Office, SESAJ-RD-SS 4400 PGA Boulevard, Ste. 500 Palm Beach Gardens, FL 33410-2933 561-472-3517	Clean Water Act	Minimum 6 months
Section 401- Water Quality Certification Refer to State/ Approvals	Fill of Wetlands	U.S. Army Corps of Engineers Ms. Tori White South Permits Section Office, SESAJ-RD-SS 4400 PGA Boulevard, Ste. 500 Palm Beach Gardens, FL 33410-2933 561-472-3517	Clean Water Act	Minimum 6 months
SPCC Plan	Petroleum Storage	EPA-Region 4	Clean Water Act	Minimum 6 months
NPDES Refer to State Permits Approvals	Wastewater Discharge	Florida Department of Environmental Protection 2600 Blair Stone Rd, MS 3560 Tallahassee, FL 33399	CWA	6-12 months

Permit/Approval	Regulated Activity	Agency and Contact	Authority	Approval Timeline*
Coastal Zone Management Consistency Refer to State Permits, Approvals	Development in Coastal County	Florida Coastal Management Program Department of Environmental Protection Mail Station #47 3900 Commonwealth Boulevard Tallahassee, FL 32399-0250 850- 245-2163	Coastal Zone Management Act	Minimum 45-60 days
Endangered Species Act Consultation	Wildlife Impacts	Florida Ecological Services Office 1339 20th Street Vero Beach, FL 32960 772-562-3909 FWC South Regional Wildlife Diversity Ricardo Zambrano Conservation Biologist 850-625-5122 FWC Imperiled Species Management/FWS West Indian Manatee (Trichechus Manatus) Mary Duncan, 850-922-4330	Endangered Species Act	3-6 months
Cultural Resources Refer to State Permits Approvals	Excavation	State Historic Preservation Office R.A. Gray Building 500 Boronough Street Tallahassee, FL 32399-0250	National Historic Preservation Act	60 days
STATE PERMITTING REQUIREMENTS				
Comprehensive Everglades Restoration Plan Act Regulation Act	Project Construction	Florida Department of Environmental Ms. Temperance M. Morgan 2600 Blair Stone Rd., MS 3560 Tallahassee, FL 32399 850-245-8424	Title XXVII Section 373, 373.1502 F.S.	12-18 months

Permit/Approval	Regulated Activity	Agency and Contact	Authority	Approval Timeline*
Well Construction	Well Construction	South Florida Water Management District 3301 Gun Club Road West Palm Beach, FL 33406 (561) 686-8800	Rule 40 E-2, F.A.C.	60-90 days
Consumptive Water Use	Water Use	South Florida Water Management 3301 Gun Club Road West Palm Beach, FL 33406 (561) 686-8800	Rule 40 E-2, F.A.C.	60-90 days
Florida Department of Transportation Access	Bridge and Road Construction	Michael Rippe, Director Southwest Area Office Florida Department of Transportation, District 1 2295 Victoria Ave, Ste# 292 Ft. Myers, FL 33901 (863) 519-2628	Section 40E-6.091, F.A.C.	60-90 days
Clean Air Construction (PSD) Clean Air Title V-Operating Permit	Pump Station Emissions	Florida Department of Environmental Protection, Division of Air Response Management 2600 Blair Stone Rd, MS 3560 Tallahassee, FL 32399-0250	Section 62-210.300, F.A.C.	60-90 days
Petroleum Storage Tanks	Storage Tank Installation	Florida Department of Environmental Protection 2600 Blair Stone Rd, MS 3560 Tallahassee, FL 32399-0250	Rules 62-761, 62-762- F.A.C.	3-6 months

Permit/Approval	Regulated Activity	Agency and Contact	Authority	Approval Timeline*
Dewatering	Dewatering	South Florida Water Management 3301 Gun Club Road West Palm Beach, FL 33406 561-686-8800	Rule 40 E-20, F.A.C.	60-90 days
NPDES	Stormwater	DEP NPDES Stormwater Program 2600 Blair Stone Rd, MS 2500 Tallahassee, FL 33399 850-245-7522	Rule 62-621 F.A.C	6-9 months
NPDES	Produced Groundwater	Florida Department of Environmental Ms. Temperance M. Morgan 2600 Blair Stone Rd., MS 3560 Tallahassee, FL 32399 850-245-8424	Rule 62-621 F.A.C.	6-9 months
Dam Safety	Embankment Construction	Florida Department of Environmental Protection 2600 Blair Stone Rd, MS 3560 Tallahassee, FL 33399	Chapter 373 F.S.	3-6 months
LOCAL PERMITTING REQUIREMENTS				
Development Review Building Permits Zoning Approval	EAA Reservoir A-1 Ancillary Facilities	Planning, Zoning, & Building Dept. Midwestern Office 200 Civic Center Way Suite 300 Royal Palm Beach, FL 33411 561-784-1300	County Ordinance	3-6 months
Vegetation Preservation and Protection Stormwater Pollution Prevention Permit	EAA Reservoir A-1 Ancillary Facilities Construction	Environmental Resource Management Florida Department of Environmental Protection 2600 Blair Stone Rd, MS 3560 Tallahassee, FL 33399	County Ordinance	3-6 months

Permit/Approval	Regulated Activity	Agency and Contact	Authority	Approval Timeline*
Wellfield Protection Petroleum Storage Tank				

* From the date of permit application submittal.

SECTION 5

HYDROLOGY

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5. HYDROLOGY

5.1 *DESIGN CRITERIA*

The evaluation of the amount of freeboard necessary to prevent overtopping of the EAA Reservoir A-1 during different wind and precipitation conditions is described in this section. The design criteria for the determination of wind speed coincident with precipitation and the normal pool level were summarized in the Design Criteria Memorandum 2 (DCM-2) (Haapala et. al., 2005a). The first design condition evaluated was a 100-year wind in combination with the PMP event. The 72-hour PMP for the EAA Reservoir A-1 was calculated to be about 54 inches (Burgi et al., 2005). The second design condition includes rainfall that would occur during a 100-year storm in combination with a category five hurricane. According to DCM-2, a one-minute wind speed of 156 miles per hour (mph) should be used for this design condition. The third design condition includes the probable maximum wind that, according to DCM-2, was 200 mph. This 200-mph wind speed was assumed to be an over-water, one-minute average wind speed. The one-minute wind speed was converted to a one-hour average wind speed of 158 mph. According to the DCM-2, the probable maximum wind condition should be used for sensitivity identification and not as a selected design condition. The fourth design condition is based on recorded data from hurricane Easy which occurred in Florida in 1950. A maximum three-second gust wind speed of 125 mph was recorded during the hurricane. This wind speed was converted using the procedures outlined in DCM-2 to an adjusted wind speed of 96 mph.

DCM-2 contains guidelines for developing the antecedent water depth at the start of the PMP event (Case 1). It included a provision for a 30 percent PMP storm, which was followed by three days of dry weather and then the PMP. The initial rain (30-percent PMP, 1.4 feet) is routed during the three dry days to determine the initial water depth at the beginning of the PMP storm. The entire EAA Reservoir A-1 30 percent PMP volume will be discharged during the three-day dry interval, which equates to approximately 4,000 cfs over those three dry days. The design inflow for STA-3/4 is 6,000 cfs, so the proposed releases of 4,000 cfs from EAA Reservoir A-1 is lower than the maximum design inflow of STA-3/4. With the addition of the 30 percent PMP direct precipitation to STA-3/4 and the 30-percent PMP release from the EAA Reservoir A-1, the depth in STA-3/4 would be less than the depth of the standard project storm of 36 inches (SFWMD, 2004). As a result, it is not anticipated that releases from EAA Reservoir A-1, including the discharge of 30 percent of the PMP over three dry days, will detrimentally impact the STA-3/4. Therefore, the additional 30-percent PMP (1.4 feet) was not added to the WSE of the EAA Reservoir A-1 when computing the EAA Reservoir A-1 depth to contain the PMP.

Considerations have also been given to storing water above the normal pool level of 12 feet in the EAA Reservoir A-1 (Hall, 2005). Three scenarios were described where four, eight, and 12 feet of additional storage were added above the normal pool level. For each scenario, it was assumed that the PMP would result from a hurricane or tropical storm and that operations personnel would have at least three days warning before the storm’s arrival. This three-day period would allow operations personnel to release sufficient water (additional 4, 8, or 12 feet) prior to the advent of such a storm to return the water level in the EAA Reservoir A-1 to its 12 foot design depth. For the four foot scenario, a discharge capability of 10,800 cfs over a 72-hour emergency draw down period was required, while the eight and 12 foot scenarios would require discharge capabilities of 21,500 cfs and 32,000 cfs, respectively. Discharge under any of these scenarios would overwhelm the capacity of the adjacent canals and STAs. The storage of additional water over the normal pool level of 12 feet is not recommended. The increased depth of water would increase seepage rates and impact the stability of the embankment. In addition, significant cost increases would result from additional pumping capacity requirements for a higher head (water level).

5.2 HAZARD CLASSIFICATION AND EMERGENCY EVACUATION REQUIREMENTS

The EAA Reservoir A-1 Embankment and Reservoir has been determined to be classified as high hazard (major impoundment), as specified in the Federal Energy Regulatory Commission's (FERC) *Selecting and Accommodating Inflow Design Floods for Dams* (FERC, 1993) and *Design Criteria Memorandum-1: Hazard Potential Classification* (Haapala et al., 2005b) guidelines. U.S. 27 carries a large traffic volume and will be located directly east of the EAA Reservoir A-1 embankment. A direct loss of life is imminent if the embankment fails, especially if the failure were to occur on the eastern side. The potential depth and velocity of a floodwave across U.S. 27 is greater than 10 feet high at 10 feet per second from a PMP breach. Furthermore, U.S. 27 is a hurricane evacuation route for residents of South Florida, so not only is an EAA Reservoir A-1 embankment failure a direct threat to motorists, an embankment failure would inhibit a major evacuation route for the surrounding population. See Section 24 for a discussion regarding the Emergency Action Plan to be developed for the EAA Reservoir A-1.

5.3 DESIGN STORMS AND FLOODS

Four wind and precipitation design conditions to be used on Acceler8 projects were developed and issued in draft form in DCM-2 (Haapala et al., 2005a). The design conditions that were modeled are described below. Additional details on developing the wind speeds and water levels to represent these design conditions are presented in Appendix 5-21, Wave Run-up Case Descriptions.

5.3.1 100-Year Wind with Probable Maximum Precipitation

The first design condition evaluated was a 100-year wind in combination with the PMP event. The 72-hour PMP for the EAA Reservoir A-1 was calculated to be about 54 inches (Appendix 5-2). Hydrometeorological Reports (HMR) No. 36, 43, 49, 51, 52 and 55 were developed to analyze data, and to provide logic and methodology for predicting the PMP for a given area (between 10 and 20,000 square miles) within the United States (NOAA, 1978, 1982). HMR51 and HRM52 are used for determining the PMP east of the 105th Meridian, including the EAA Reservoir A-1 site. The HMR52 computer program, developed by the USACE, automates the calculations used to follow the procedures in HMR52. HMR52 recommends a procedure for estimating the PMP in an area for which both a temporal and a spatial distribution of the precipitation are required (USACE, 1984). Utilizing this program and the EAA Reservoir A-1 embankment footprint under consideration, a calculated value of 53.54 inches of rain for the 72-hour PMP was developed.

The procedure described in DCM-2, (Haapapa et al., 2005a) was followed to determine the 100-year wind for the EAA Reservoir A-1. According to Figure DCM-2-2, the 50-year, three second wind gust for the EAA Reservoir A-1 site is 125 mph. This number was converted to a

100-year one-hour wind speed of 107 mph. After adjustments for duration and overwater conditions, the final wind speed to represent this design condition was calculated to be 103 mph.

5.3.2 Category Five Hurricane with 100-Year storm

The second design condition includes rainfall that would occur during a 100-year storm in combination with a category five hurricane. According to DCM-2, a one-minute wind speed of 156 mph should be used for this design condition. After adjustments for duration, the final wind speed to represent this design condition was 122 mph. Using Figure DCM-2-3 it was determined that the appropriate rainfall for this condition is 17 inches at the EAA Reservoir A-1.

5.3.3 Probable Maximum Wind (200 mph)

The third design condition includes the probable maximum wind that, according to DCM-2, was 200 mph. The DCM-2 states the following about probable maximum wind condition:

> [The probable maximum wind…] is to be used for "sensitivity identification" and not as a design condition. Wave models are unlikely capable of yielding results within a degree of confidence for design for these extreme wind speeds, especially over relatively shallow water bodies. Even for 125-mph wind, model capabilities are most likely being "stretched" for project conditions.

Therefore, this design condition was evaluated but was not selected as the critical design condition used to size the embankment. This 200-mph wind speed was assumed to be an over-water, one-minute average wind speed. The one-minute wind speed was converted to a one-hour average wind speed of 161 mph. Using Equation 4 of DCM-2 and considering the fetch of the EAA Reservoir A-1, the 161-mph wind speed was converted to 158 mph. It was assumed that this wind would occur with the EAA Reservoir A-1 at the normal maximum operating level. The normal maximum water level for the EAA Reservoir A-1 is expected to be about 12 feet. A normal maximum operating level of 12 feet was used in the modeling.

5.3.4 Storm Specific Wind and Precipitation

The fourth design condition is based on recorded data from hurricane Easy which occurred in Florida in 1950. A maximum three-second gust wind speed of 125 mph was recorded during the hurricane. This wind speed was converted using the procedures outlined in DCM-2 to a final wind speed of 96 mph. During hurricane Easy, a peak 24-hour rainfall total of 38.7 inches was recorded. For this design condition, a wind speed of 96 mph was applied to the EAA Reservoir A-1 at a water depth of 15.2 feet. Because the wind speed and water depth for design condition four are both less than those of design condition one, the required freeboard for this design condition would be less than that required under design condition one. Therefore, this condition received no further consideration.

5.4 EAA Reservoir A-1 Inflows and Outflows

The EAA Reservoir A-1 has a normal maximum pool depth of approximately 12 feet or a maximum WSE of 20.6 feet NAVD88. The average bottom elevation is 8.6 feet NAVD88.

5.4.1 Inflow Design Storm

The inflow design storm (IDF) for the EAA Reservoir A-1 will be the probable maximum flood (PMF) as designated by DCM-2. Because the EAA Reservoir A-1 functions as an off-line reservoir and has no contributing watershed except for its surface area, the PMF is the PMP depth of 4.5 feet distributed appropriately in time. To determine the maximum inflow, the maximum precipitation rate is multiplied by the area of the EAA Reservoir A-1 site (inflow pumps are assumed not to be operating at the time). Utilizing the results of the HMR52 model as described in Appendix 5-1, the maximum inflow to EAA Reservoir A-1 from the 72-hour PMP precipitation event would be approximately 288,000 cfs corresponding to a precipitation depth of 1.5 inches falling in five minutes across the entire EAA Reservoir A-1. Appendix 5-7 shows the entire inflow hydrograph for each of the five PMP runs generated.

5.4.2 Routing of Flood Flows

Because the EAA Reservoir A-1 is perched on all sides and has no contributing watershed except for the surface area of the EAA Reservoir A-1, there are no direct gravity inflows. The EAA Reservoir A-1 will be fitted with several gate structures capable of routing significant flood flows. See Section 6 for detailed discussion regarding gate structures. During storm events, the EAA Reservoir A-1 will be capable of passing flow to the downstream areas which include the NNRC and Miami Canal and the STA-3/4 Supply Canal. However, as defined in DCM-2, during the PMP event which is used, in part, to determine the maximum freeboard requirements, no reservoir routing is assumed to take place while the PMP is occurring. It is anticipated that the gates will be inoperable during the PMP making the gate routing irrelevant. In other words, the EAA Reservoir A-1 must be designed to be capable of containing the full PMP/PMF storm because reservoir routing is assumed to not be applicable.

DCM-2 guidance states that 30 percent of the PMP will fall followed by three dry days in which reservoir routing can take place before the actual PMP event occurs (for freeboard determination). As discussed in Section 5.1, this 30-percent PMP is completely routed via the gate structures before the start of the PMP event.

An uncontrolled spillway capable of routing significant flood flows was considered for the EAA Reservoir A-1. However, due to downstream limitations, detailed in Section 6, the selected EAA Reservoir A-1 uncontrolled spillway configuration will have a negligible effect on flood routing and Reservoir A-1drawdown during the Inflow Design Flood (IDF) event.

5.4.3 EAA Reservoir A-1 Discharges

Discharges from the EAA Reservoir A-1 will be based on expected environmental deliveries for the EAA Reservoir A-1 and agricultural deliveries for the NNRC/Hillsboro Canal basin. Discharge structures include gates that will be sized according to the flows released from the EAA Reservoir A-1 per the SFWMD ECP 2010 and ECP 2015 (version 5.4.2) simulations. Gate discharges will follow orifice flow principles and gate openings will be a function of the releases from the EAA Reservoir A-1.

5.5 FREEBOARD/SUPERIORITY

Wave run-up modeling was conducted to determine the amount of freeboard required to prevent over-topping of the EAA Reservoir A-1 embankment during high wind and rain conditions and to determine the effectiveness of internal breakwaters in decreasing wave run-up. The Automated Coastal Engineering System (ACES) model was used to conduct this analysis. Details of the wave run-up modeling are provided in Appendices 5-16 and 5-17.

In addition to the three design conditions modeled, two embankment types (zoned embankment and RCC) were simulated as well as the effects of a perimeter bench. Characteristics for the zoned embankment included 3H:1V side slopes and a rough surface. Roughness coefficients for rip-rap were used in the modeling. Characteristics for the RCC dam included a vertical wall with a smooth surface. In all cases it was assumed that the perimeter bench would have a 3H:1V slope and a rough surface. Bench widths of 25 feet and 15 feet were simulated.

Modeling conducted prior to the issuance of DCM-2 examined the effects of variations in fetch distance, water depth, slope, and surface roughness on wave growth and run-up. The effectiveness of internal breakwaters to reduce wave run-up and maximum water level was also examined.

The ACES program was used to calculate wave growth, wave run-up, and wave transmission over the perimeter bench. The ACES model does not calculate wind set-up and this was calculated separately using the Sibul model (USACE, 2004). Additional information on the model configuration, model calibration, verification, and reliability is provided in Appendices 5-16, 5-17, and 5-18.

5.5.1 Wave Characteristics

The wave growth section of the ACES model was used to identify the wave characteristics that could occur under the design conditions. Wave growth is a function of the speed and duration of winds, fetch distance, and water depth. The wave height and wave period increase with increasing fetch, depth and wind speed. The effective depth was used to generate wave characteristics including wave height. Wave heights are included on Table 5.5-1 and ranged from 6.5 to 7.1 feet for the design conditions modeled. The other results presented in the table are discussed in subsequent sections.

Table 5.5-1 Wave Run-Up Results

Wind Speed (mph)[1]	Wave Height (feet)	Rainfall Depth (feet)	Effective Depth (feet)	Wave Run-Up (feet)	Wind Set-up (feet)	Maximum Water Level (feet)	Embankment Height (feet)
Zoned Embankment, 3H:1V Slope, Rough Surface							
103	6.65	4.5	16.5	6.0	2.1	24.6	25.5
122	6.53	1.4	13.4	6.1	3.6	23.1	24.0
158	7.06	0.0	12.0	6.7	7.0	25.7	27.5
RCC Dam, Vertical Slope, Smooth Surface							
103	6.65	4.5	16.5	7.9	2.1	26.5	27.5
122	6.53	1.4	13.4	7.8	3.6	24.8	26.0
158	7.06	0.0	12.0	8.5	7.0	27.5	28.0
[1] mph = miles per hour, Embankment Height is distance above original ground							

5.5.2 Wind Set-up

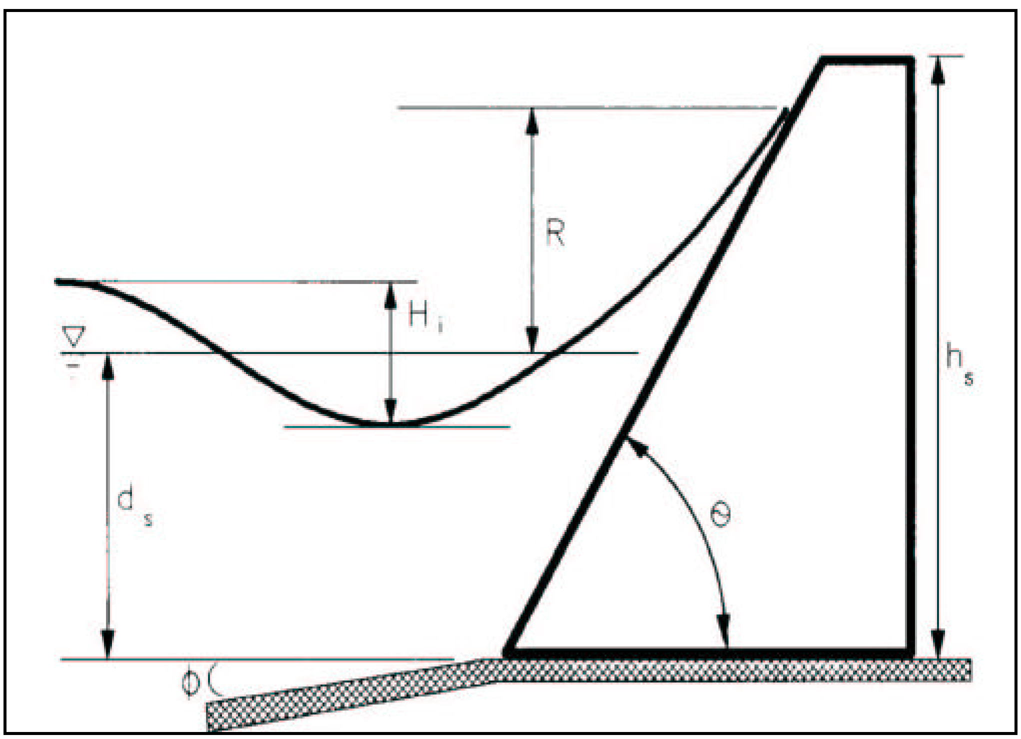
Wind set-up can be an important factor in determining freeboard requirements. Wind set-up occurs when wind blows in a relatively constant direction over the water surface. Shear stresses between the wind and water exert a drag on the water and push the water in the direction of the wind. When the water encounters a barrier such as a shoreline or embankment it piles up resulting in deeper water at the shoreline. Wind set-up will increase until there is a balance between the shear stresses on the water surface and a gravity induced return flow along the EAA Reservoir A-1 bottom. Wind set-up is a function of wind speed, fetch, and water depth. Wind set-up increases with wind speed and fetch but decreases with increasing water depth.

Wind set-up is not included in the ACES model. The Sibul model was used to calculate wind set-up and the results were added to the wave run-up calculations. Wind set-up calculations were made for each of the cases evaluated and results are included on Table 5.5-1 . Wind set-up increases with increasing wind speed and fetch. Wind set-up decreases with increasing depth. Wind set-up for the design conditions ranged from 2.1 to 7.0 feet.

5.5.3 Wave Run-up

The wave run-up section of the ACES model calculates the run-up that occurs when waves encounter a shoreline or embankment. The required inputs include wave type, breaking criteria, wave height, wave period, structure slope, structure height, slope type, and roughness coefficient. This section of the model also calculates overtopping rates. The output calculated by the model includes wave run-up, deepwater wave height, and wave steepness. Figure 5.5-1 indicates how the wave run-up parameters are defined. Wave run-up (R) is measured from the still water level as opposed to wave height (H), which is measured from trough to crest. The normal water level is d_s and its embankment height is h_s.

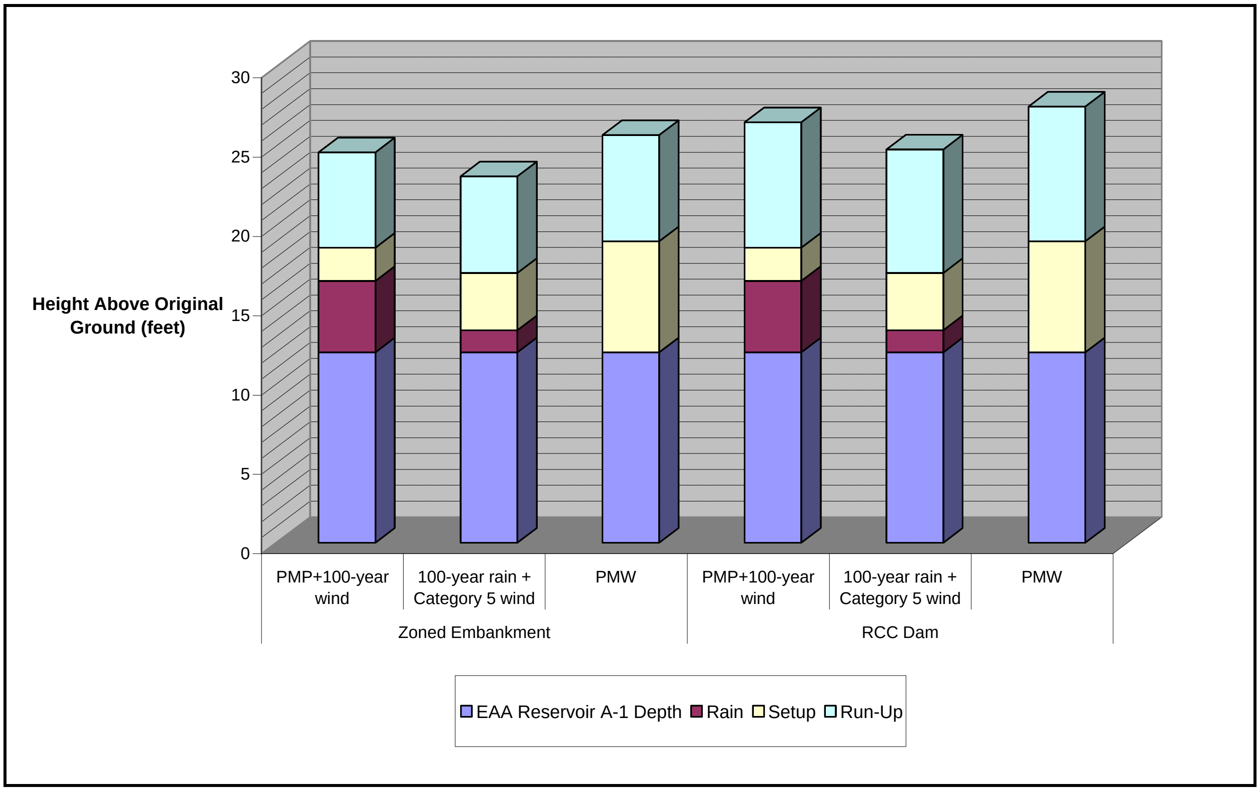
Figure 5.5-1 Definition of Wave Run-Up Parameters



(Leenknecht et al., 1992)

The wave-run-up module of the ACES model was used to estimate wave run-up for each of the design conditions evaluated. The results of the wave run-up computer modeling are presented in Table 5.5-1 . This table lists the wind speed, wave height, rainfall amount, effective depth, wave run-up, wind set-up, and maximum water level. The maximum water level is the sum of the effective depth, wind set-up and wave run-up. Figure 5.5-2 provides a graphic representation of the elements included in the maximum water level.

Figure 5.5-2 Elements Included in the Maximum Water Level



5.5.4 Effects of Internal Breakwaters

Two configurations of internal breakwaters were evaluated; a peripheral wall located approximately 0.5 mile inside of the embankment, and a circle breakwater in the middle of the EAA Reservoir A-1 with several spokes radiating toward the embankments. The results of the modeling indicated that the peripheral wall would allow reduction of the embankment height by at least four to seven feet. The circle breakwater would not be as effective at reducing freeboard and may reduce the embankment height by only about one foot. These structures would be very large and would not be cost-effective. Details on the evaluation of the internal breakwaters are presented in Appendix 5-16.

5.5.5 Effects of Perimeter Bench

Modeling was also conducted to determine the effectiveness of a perimeter bench on reducing wave run-up, thereby, reducing freeboard requirements. A perimeter bench would require significantly less material to construct than the internal breakwaters. Modeling conducted by the

USACE for the C-43 Reservoir (Hadley, 2005) showed that a 25-foot wide perimeter bench would break waves and could significantly reduce the required freeboard. The C-43 Reservoir modeling showed that a bench submerged at a depth of three feet below the maximum surcharge depth would reduce wave heights to about one third of the incident wave height and reduce wave periods by about 10 percent.

The ACES model can not simulate the effects of a submerged bench. With the wind set-up, water depth could be as high as 19 feet before adding wave run-up. For modeling purposes, the bench was set at a depth of 19.05 feet. It is recognized that a bench at a lower depth, such as 16 feet, would be just as effective. Modeling to optimize the bench depth, width and configuration will be conducted during preliminary design.

The bench was simulated in the ACES program by first modeling the incident wave on an impermeable breakwater with a height of 19 feet, a width of 25 or 15 feet, with 3H:1V side slopes covered with rip-rap. The transmitted wave characteristics were then used as the wave characteristics that would run-up on the embankment.

Results of the cases that include a 25 foot bench are presented in Table 5.5-2 . The bench has a 3H:1V slope, a width of 25 feet, and is 19 feet high, above original ground level. The transmitted wave height and period describe the characteristics of the wave that would be running up on the embankment. It is possible that the need for rip-rap covering the embankment could reduce the width of the bench. Additional cases for a 15-feet wide bench for both the Zoned and RCC dams were modeled and the results are presented in Table 5.5-3

A perimeter bench would be very effective in reducing wave run-up on the embankment. For the two cases where the water depth is at the approximate height of the bench, the transmitted wave is about one third the height of the incident wave. This is approximately the same ratio calculated by the USACE in their modeling of the C-43 Reservoir. A submerged bench would also be effective in breaking the incident wave and reducing wave run-up.

Table 5.5-2 Results of Cases With a 25-Foot Bench

Wind (miles per hour)	Embankment Slope, Surface	Water Depth [a] (feet)	Transmitted Wave		Wave Run-Up (feet)	Maximum Water Level [b] (feet)
			Height (feet)	Period (seconds)		
103	3H:1V, rough	18.6	2.27	4.6	2.6	21.6
122	3H:1V, rough	17.0	1.60	4.8	2.1	21.1
158	3H:1V, rough	19.0	2.55	5.2	3.0	22.0
103	vertical, smooth	18.6	2.27	4.6	2.5	21.5
122	vertical, smooth	17.0	1.60	4.8	1.8	20.8
158	vertical, smooth	19.0	2.55	5.2	2.9	21.9

a: Water depth is the sum of the normal maximum level, rainfall, and the wind set-up
b: Wave run-up heights were added to the 19-foot bench depth

Table 5.5-3 Results of Cases with a 15-Foot Bench

Wind (miles per hour)	Embankment Slope, Surface	Water Depth [a] (feet)	Transmitted Wave		Wave Run-Up (feet)	Maximum Water Level [b] (feet)
			Height (feet)	Period (seconds)		
103	3H:1V, rough	18.6	2.27	4.6	2.9	21.9
122	3H:1V, rough	17.0	1.60	4.8	2.3	21.3
158	3H:1V, rough	19.0	2.55	5.2	3.4	22.4
103	vertical, smooth	18.6	2.27	4.6	3.0	22.0
122	vertical, smooth	17.0	1.60	4.8	2.1	21.1
158	vertical, smooth	19.0	2.55	5.2	3.4	22.4

a: Water depth is the sum of the normal maximum level, rainfall, and the wind set-up
b: Wave run-up heights were added to the 19-foot bench depth

5.5.6 Overtopping Analysis

The ACES model was also used to calculate overtopping rates for the three design cases and both embankment types. Overtopping rates were calculated in one-foot increments starting at the Maximum Water Level and continuing until the overtopping rate was less than 0.1 cfs per lineal foot of embankment. According to DCM-2 (Haapala et al., 2005a), zero over-wash is defined as 0.1 cfs per lineal foot of embankment length for an exterior earthfill slope. The maximum water level is the sum of the effective depth, wind set-up and wave run-up. At this level there would be no overtopping for a monochromatic wave field. An overtopping rate was not calculated (NC) for any case where the embankment height was less than the maximum water level.

The overtopping analysis was conducted assuming irregular waves. This recognizes that wind generated waves are not uniform and that a small percentage of waves will run-up higher onto the embankment than the predicted height. Table 5.5-4 presents the overtopping analysis for the embankments without a perimeter bench. Table 5.5-5 and Table 5.5-6 present the overtopping analysis for embankments with a 25-foot wide and 15-foot wide perimeter bench, respectively. Using the results of the overtopping analysis the required embankment height was determined for each case. These results are included on Table 5.5-1.

Table 5.5-4 Results of Overtopping Analysis Cases Without a Bench ($feet^3$/second/feet)

Embankment Height (feet)	158 miles per hour		122 miles per hour		103 miles per hour	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
24	NC[1]	NC	0.09	NC	NC	NC
25	NC	NC		0.27	0.15	NC
26	0.270	NC		0.104	0.001	NC
27	0.105	NC		0.041		0.146
28	0.037	0.027				0.060

[1]NC - a value was not calculated because the embankment height was less than the Maximum Water Level
$feet^3$/second/feet = cubic feet per second per linear foot

Table 5.5-5 Results of Overtopping Analysis Cases with a 25-Foot Wide Bench ($feet^3$/second/feet)

Embankment Height (ft)	158 miles per hour		122 miles per hour		103 mph	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
21	NC	NC	NC	0.08	NC	NC
22	0.086	0.081	0.001		0.017	0.013

$feet^3$/second/feet = cubic feet per second per linear foot

Table 5.5-6 Results of Overtopping Analysis Cases with a 15-Foot Wide Bench ($feet^3$/second/feet)

Embankment Height (ft)	158 miles per hour		122 miles per hour		103 miles per hour	
	Zoned	RCC	Zoned	RCC	Zoned	RCC
22	NC	NC	0.005	0.001	0.037	0.043
23	0.036	0.039	0.000	0.000	0.004	0.004

NC - a value was not calculated because the embankment height was less than the Maximum Water Level
$feet^3$/second/feet = cubic feet per second per linear foot

5.5.7 Summary

Wave run-up modeling was conducted to determine the amount of freeboard required to prevent over-topping of the EAA Reservoir A-1 embankment during high wind and rain conditions and to determine the effectiveness of internal breakwaters in decreasing wave run-up. The available freeboard should be sufficient to contain the EAA Reservoir A-1 at the maximum normal operating level in addition to rainfall, wind set-up, and wave run-up that could occur. Wave run-up modeling was conducted to simulate the design conditions specified in DCM-2.

Wave heights, wind set-up and wave run-up were calculated for each design condition and for two embankment types. Wave heights for the design conditions ranged from 6.5 to 7.1 feet. Wind set-up for the design conditions ranged from 2.1 to 7.0 feet. The wave run-up module of the ACES model was used to estimate wave run-up for each of the design conditions evaluated. The maximum water level is the sum of the effective depth, wind set-up and wave run-up. The maximum water level ranges from 23.1 to 25.7 feet for the zoned embankment and from 24.8 to 27.5 feet for the RCC dam.

Modeling was also conducted to determine the effectiveness of a perimeter bench on wave run-up thereby reducing the required freeboard. A perimeter bench would be very effective in reducing wave run-up on the embankment. For modeling purposes, the bench was set at a depth of 19.05 feet. For a 25-foot wide bench the maximum water level ranges from 21.1 to 22.0 feet for the zoned embankment and from 20.8 to 21.9 feet for the RCC dam. For a 15 feet wide bench the maximum water level ranges from 21.3 to 22.4 feet for the zoned embankment and from 21.8 to 22.4 feet for the RCC dam. A submerged bench would also be effective in breaking the incident wave and reducing wave run-up.

The embankment height should be set higher than the Maximum Water Level (MWL) to prevent overtopping to account for the irregular nature of waves. Overtopping rates were calculated in one foot increments starting at the MWL and continuing until the overtopping rate was less than 0.1 cfs per lineal foot. Without a perimeter bench, the embankment height would need to be about 26 feet above the EAA Reservoir A-1 bottom for a zoned embankment and about 28 feet above the EAA Reservoir A-1 bottom for an RCC dam to prevent overtopping. The overtopping analysis indicates that the embankment height can be significantly reduced with the addition of a perimeter bench. With a perimeter bench, an embankment height of about 22 feet above the EAA Reservoir A-1 bottom would prevent overtopping for both types of embankments.

The costs associated with these alternatives are discussed in Section 8.

5.6 REFERENCES

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SECTION 6

WATER STORAGE AND RESERVOIR OPTIMIZATION

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6. WATER STORAGE AND RESERVOIR OPTIMIZATION

6.1 INTRODUCTION

This Section of the BODR describes the methods used to determine environmental and agricultural deliveries, EAA Reservoir A-1 seepage, flows in the seepage canals, water quality in the EAA Reservoir A-1, and the water control structures (pump station, gates and spillway). This introduction summarizes the more detailed discussions presented later in this Section.

6.1.1 Environmental and Agricultural Deliveries

The environmental deliveries via STA-3/4 and the specific agricultural deliveries to be supplied by the EAA Reservoir A-1 were provided by the OoM, based on the SFWMM ECP 2010 and ECP 2015 simulations (version 5.4.2). This is the same model (including version number) as the ECP 2010 and 2015 runs for the Regional Feasibility Study. The conditions and assumptions of the ECP 2010 and ECP 2015 simulations vary depending on land use and which Acceler8 projects are implemented. The environmental deliveries from the ECP 2015 simulation were used to maximize the supply of these deliveries from the EAA Reservoir A-1 based on storage capacity, while the ECP 2010 agricultural deliveries and other available flows were used from the ECP 2010 simulation. Based on discussions with OoM and the EAA Reservoir A-1 Project team, the environmental deliveries from the ECP 2015 simulation were distributed using a four-day moving average over the POS, which resulted in lower peak environmental deliveries and extended the deliveries over the POS. The POS was 1965 to 2000, or 36 years. The results of the ECP 2015 simulation indicate that the average annual environmental delivery from the EAA Reservoir A-1 via STA-3/4 is approximately 901,000 acre-feet, with a maximum of 2,256,000 acre-feet in water year 1970, and a minimum of 104,000 acre-feet in water year 1990. The current average annual inflow into STA-3/4 is approximately 656,000 acre-feet (Piccone, 2005). The total delivery over the POS is approximately 31,788,000 acre-feet. These values are the environmental calls simulated in the ECP 2015 run and not the deliveries supplied by the EAA Reservoir A-1 using the WBM.

The results of the ECP 2010 simulation indicate that the average annual agricultural delivery from the EAA Reservoir A-1 to the NNRC/Hillsboro Canal basin is approximately 129,000 acre-feet, with a maximum of 215,000 acre-feet in water year 2000, and a minimum of 77,000 acre-feet in water year 1969. The total delivery over the POS is approximately 4,756,000 acre-feet. These values are the agricultural calls simulated in the ECP 2010 run and not the deliveries supplied by the EAA Reservoir A-1 using the WBM.

6.1.2 Pump Station

A new pump station in the northeast corner of the EAA Reservoir A-1 is recommended to pump water from the NNRC into the EAA Reservoir A-1. Ten alternatives were evaluated. The recommended alternatives considers a new 3,600 cfs pump station and no modifications to existing G-370 and G-372 pump stations. The existing G-370 and G-372 pump stations would pump to an elevation 16.6 NAVD88 (8-foot water depth). The new pump station would pump to the full 12-foot operating depth.

6.1.3 Gates

A series of discharge gates will be located along the NNRC to control flows back to the NNRC. Another series of discharge gates will be located along the STA-3/4 Supply Canal to control flows to STA-3/4. The culvert/gate structures will be fully submerged with inverts 10 feet below the bottom of the EAA Reservoir A-1. Each of the culvert/gate structures is a series of 10 foot x 10 foot culverts with roller gates on the external side of the embankment.

6.1.4 Spillway

An orifice type spillway is recommended to provide an uncontrolled discharge from the EAA Reservoir A-1. The spillway will include a 55 foot long overflow weir that will pass approximately 500 cfs at 2 foot of depth. The spillway will discharge to a 5.5 foot square culvert that will function as an orifice to limit the flow to 650 cfs at 22 feet of head.

6.2 EAA RESERVOIR A-1 OPTIMIZATION

6.2.1 Characteristics

The site boundary of the EAA Reservoir A-1 was determined from aerial photography based on the land acquired by the SFWMD. Some assumptions were necessary to develop a preliminary stage/area/storage relationship for the EAA Reservoir A-1. They are as follows:

- A seepage canal will be required along the east and north boundaries and the northern half of the west boundary. No seepage canal will be provided along the south boundary and southern half of the west boundary where the embankment is adjacent to the STA 3/4 supply canal.
- The outside toe of the EAA Reservoir A-1 will begin approximately 425 feet in from the site boundary along the north boundary and along 4,600 feet on the north Section of the west boundary. On the east boundary, the toe will begin approximately 275 feet west of U.S. 27, on the south boundary and the rest of the west boundary, the toe will begin at the site boundary.
- A 26-foot tall embankment (above original grade or OG) will be sufficient to meet the volume, freeboard, and wave run-up requirements.
- 3H:1V side slopes with a top width of 14 feet will meet the stability requirements for the EAA Reservoir A-1 embankment.

6.2.2 Modeling

A water balance analysis was performed for the EAA Reservoir A-1 to assess the hydrologic and hydraulic components of the system. The analysis was performed with the water balance model (WBM), which was developed to analyze the EAA Reservoir A-1's storage capacity and operations on a daily basis (time step). The POS extends for 36 years from January 1, 1965 to December 31, 2000.

The WBM was used to optimize the storage capacity of the EAA Reservoir A-1, while evaluating the impacts on flows in the NNRC, Miami Canal, and the STA-3/4 Supply Canal. In addition, the model was used to evaluate pump facility locations and the distribution of releases from the EAA Reservoir A-1 for environmental and agricultural purposes.

The WBM includes the following hydrologic components:

- Direct precipitation into the EAA Reservoir A-1 (P)
- Inflow through pumps and weirs from the canals (I)
- Outflow through weirs and culverts into the canals (O)

- Net evaporation from the EAA Reservoir A-1 surface (E)
- Seepage losses (S)
- Change in storage in the EAA Reservoir A-1 (ΔS)
- The basic water balance equation is: $\Delta S = P + I - O - E - S$. This equation accounts for the change in storage in the EAA Reservoir A-1 based on inflows and outflows and is applied to the WBM on a daily basis.

Seven different water balance models were developed to evaluate different conditions of water sources and deliverables. The results from the final model are described in this section on Reservoir Optimization.

The final WBM evaluated for the EAA Reservoir A-1 used both SFWMM ECP 2010 and 2015 (version 5.4.2). The model includes the EAA Reservoir A-1 with the northeast pump station pumping to 12 feet of EAA Reservoir A-1 depth at a rate of 3,600 cfs, and G-370 and G-372 pump stations not modified and pumping to 8 feet of EAA Reservoir A-1 depth at a rate of 2,340 cfs and 3,120 cfs, respectively. EAA Reservoir A-1 outflows to STA-3/4 have been capped at 6,000 cfs to approximate the capacity of the facility's intake structures. Model inflows, evaporation outflows, and agricultural deliveries were obtained from the SFWMM ECP 2010 simulation and the environmental deliveries were obtained from the SFWMM ECP 2015 simulation. Results from the simulations were provided by the SFWMD's OoM.

6.2.3 Inflow and Outflow Data

The main input parameters into the WBM include precipitation, EAA Reservoir A-1 Project inflows and outflows, evaporation, and seepage. These parameters may be separated into Inflows and Outflows of the EAA Reservoir A-1. Data on the parameters were provided by the OoM and the USACE Interagency Modeling Center (IMC), based on simulations using the SFWMM.

The OoM provided the available flows in the NNRC and the Miami Canal, as well as the required environmental and agricultural deliveries to be supplied by the EAA Reservoir A-1, based on the SFWMM ECP 2015 and ECP 2010 simulations. The IMC provided evaporation and precipitation data based on the inputs into the SFWMM.

Based on discussions with OoM and the EAA Reservoir A-1 Project team, the Data Storage System (DSS) flow tags, listed in Table 6.2-1 and illustrated in Figure 6.2-1, were identified for the SFWMM ECP 2010 and ECP 2015 version 5.4.2, simulations. The definition of each tag is provided in Table 6.2-2.

Table 6.2-1 DSS Flow Tags for SFWMM ECP 2010 and ECP 2015 Simulations

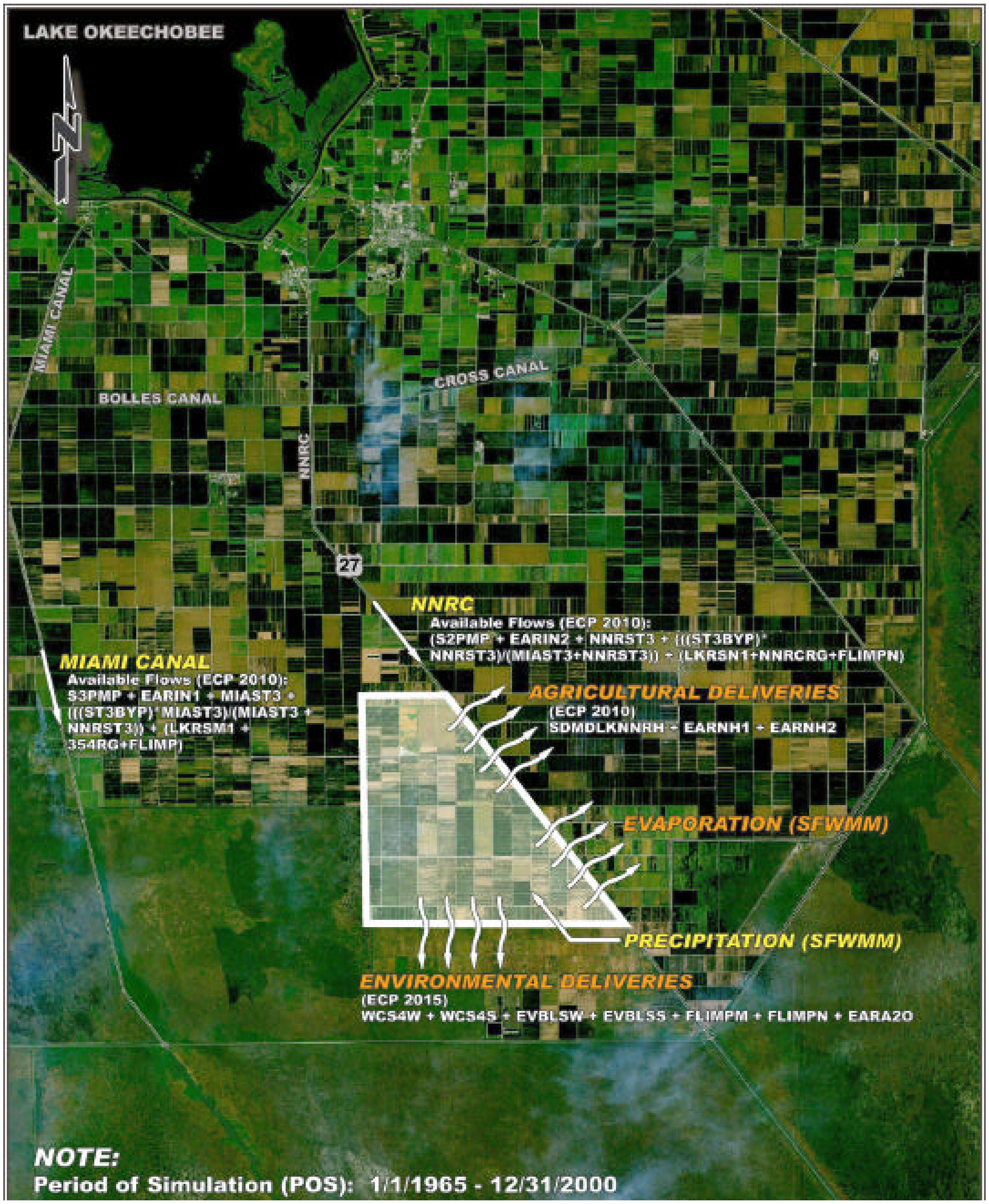
Runoff Flows	Miami Canal Basin	NNRC Basin
Runoff to Lake Okeechobee	S3PMP	S2PMP
Runoff to EAA Reservoir A-1	EARIN1	EARIN2
Runoff to STA-3/4	MIAST3	NNRST3
STA-3/4 By-pass	((ST3BYP)*MIAST3)/(MIAST3 +NNRST3)	((ST3BYP)*NNRST3)/(MIAST3+ NNRST3)
Available Flows from Lake Okeechobee	LKRSM1 + 354RG + FLIMPM	LKRSN1 + NNRCRG + FLIMPN
Agricultural Deliveries	SDMDLKMIA* + EARMA1 + EARMA2	SDMDLKNNRH* + EARNH1 + EARNH2 + EARWPB**
Environmental Deliveries	WCS4W** + WCS4S + EVBLSW** + EVBLSS + EARA2O** + FLIMPM + FLIMPN	
*Not a DSS tag **Tag may only apply to the ECP 2015 simulation		

Table 6.2-2 Definitions of Tags

Tag Name	Description
354RG	Lake Okeechobee regulatory discharge via S354.
EARA2O	Outflow from Compartment 2 of the EAA Reservoir A-1 and A-2 to WCA-2A via STA-2 for environmental water supply purposes.
EARIN1	Inflow into Compartment 1 of the EAA Reservoir A-1 and Compartment 3 of the EAA Reservoir A-1 and A-2 from the Miami Canal basin runoff.
EARIN2	Inflow into Compartment 1 of the EAA Reservoir A-1 and EAA Reservoir A-1 and A-2 from the NNRC basin runoff.
EARMA1	Outflow from Compartment 1 of the EAA Reservoir A-1 and Compartment 3 of the EAA Reservoir A-1 and A-2 to meet the Miami Canal basin supplemental agricultural deliveries.
EARMA2	Outflow from Compartment 1 of the EAA Reservoir A-1 and Compartment 3 of the EAA Reservoir A-1 and A-2 to meet the Miami Canal basin supplemental agricultural deliveries not met by EARMA1.
EARNH1	Outflow from Compartment 1 of the EAA Reservoir A-1 and the EAA Reservoir A-1 and A-2 to supply the NNRC/Hillsboro Canal basin supplemental agricultural deliveries.
EARNH2	Outflow from Compartment 1 of the EAA Reservoir A-1 and the EAA Reservoir A-1 and A-2 to supply the NNRC/Hillsboro Canal basin supplemental agricultural deliveries not met by EARNH1.
EARWPB	Outflow from Compartment 1 of the EAA Reservoir A-1 and A-2 to supply the West Palm Beach Canal basin supplemental agricultural deliveries.
EVBLSS	Subsurface water outflow down to 1.5 feet below land surface from Compartment 2 of the EAA Reservoir A-1 and the EAA Reservoir A-1 and A-2 to WCA-3A via STA-3/4 for environmental water supply purposes.
EVBLSW	Subsurface water outflow down to 1.5 feet below land surface from Compartment 4 of the EAA Reservoir A-1 and A-2 to WCA-3A via STA-3/4 for environmental water supply purposes.
FLIMPM	Import Glades water met by Lake Okeechobee via the Miami Canal through S-354.

Tag Name	Description
FLIMPN	Import Glades water met by Lake Okeechobee via the NNRC through S-351.
LKRSM1	Excess water from Lake Okeechobee via the Miami Canal to Compartment 2 of the EAA Reservoir A-1 and Compartment 4 of the EAA Reservoir A-1 and A-2.
LKRSN1	Excess water from Lake Okeechobee via the NNRC to Compartment 2 of the EAA Reservoir A-1, and the EAA Reservoir A-1 and A-2.
MIAST3	Runoff from Miami Canal basin, Chapter 298 F.A.C. Drainage District, S-236 basin, and G-136 pump station to STA-3/4 through Miami Canal and G-372 pump station.
NNRCRG	Lake Okeechobee regulatory discharge via the NNRC.
NNRST3	NNRC basin runoff routed to STA-3/4 through the NNRC and G-370 pump station.
S2PMP	Backpumping of runoff from the NNRC/Hillsborough Canal basin to Lake Okeechobee via S-2.
S3PMP	Flow backpumped for flood control into Lake Okeechobee from the Miami Canal basin.
SDMDLKMIA	Supplemental agricultural deliveries in the Miami Canal basin (including Sugar Ranch) to be met by Lake Okeechobee.
SDMDLKNNRH	Supplemental agricultural deliveries in the NNRC/Hillsboro Canal Basin to be met by Lake Okeechobee.
ST3BYP	Volume of EAA runoff that bypasses STA-3/4 untreated into WCAs.
WCS4S	Surface water outflow from Compartment 2 of the EAA Reservoir A-1 and the EAA Reservoir A-1 and A-2 to WCA-3A via STA-3/4 for environmental water supply purposes.
WCS4W	Surface water outflow from Compartment 4 of the EAA Reservoir A-1 and A-2 to WCA-3A via STA-3/4 for environmental water supply purposes.
Note: STA = Stormwater Treatment Area, WCA = Water Conservation Area, EAA = Everglades Agricultural Area, NNRC = North New River Canal	

Figure 6.2-1 Data Storage System (DSS) Flow Tags



6.2.4 Inflow

EAA Reservoir A-1 inflows in the WBM consist of flows from the NNRC, STA-3/4 Supply Canal west, seepage collection canals, and precipitation. A description of each inflow is provided below.

- NNRC Inflow that is available from the NNRC. This value includes runoff flows, Lake Okeechobee releases and Lake Okeechobee pump backs, based on the OoM ECP 2010 simulation. The available flow from the NNRC is equal to:

  Available Flow from the NNRC = S2PMP + EARIN2 + NNRST3 + ((ST3BYP) NNRST3)/(MIAST3+NNRST3) + (LKRSN1 + NNRCRG + FLIMPN)*

  Based on the ECP 2010 simulation, the average yearly available flow in the NNRC over the POS is approximately 379,000 acre-feet, with a maximum of 640,000 acre-feet in water year 1980, and a minimum of 131,217 acre-feet in water year 1989.

  Based on the WBM, the average yearly inflow into the EAA Reservoir A-1 from the NNRC over the POS is approximately 362,000 acre-feet, with a maximum of 640,198 acre-feet in water year 1980 and a minimum of 129,000 acre-feet in water year 1989.

- Miami Canal Inflow is the flow from the Miami Canal. This includes runoff flows, Lake Okeechobee releases, and Lake Okeechobee pump backs based on the OoM ECP 2010 simulation. The available flow from the Miami Canal is equal to:

  *Available Flow from the Miami Canal = S3PMP + EARIN1 + MIAST3 + ((ST3BYP) * MIAST3)/(MIAST3+NNRST3) + (LKRSM1 + 354RG + FLIMPM)*

  Based on the ECP 2010 simulation, the average yearly available flow in the Miami Canal over the POS is approximately 499,000 acre-feet, with a maximum of 889,000 acre-feet in 1970 and a minimum of 123,000 acre-feet in 1990.

  Based on the WBM, the average yearly inflow into the EAA Reservoir A-1 from the Miami Canal over the POS is approximately 372,000 acre-feet, with a maximum of 838,000 acre-feet in water year 1970 and a minimum of 40,166 acre-feet in water year 1982.

- Collected Seepage is the seepage flows from the EAA Reservoir A-1 collected in the seepage canals. Based on the seepage analysis work performed during the Test Cells Program, the collected seepage was found to be a function of the EAA Reservoir A-1 water depth and seepage reduction alternative selected. For a scenario with an embankment that includes a 34 foot deep seepage cutoff wall and a 20 foot deep seepage canal, the collected seepage may be approximated with the polynomial equation:

  $$\text{Collected Seepage} = 0.0012x^2 - 0.0464x + 1.0752$$

  Where:

  - Collected Seepage is expressed as the percentage of the total seepage from the EAA Reservoir A-1 collected by the seepage canals, and
  - *x* is the EAA Reservoir A-1 water depth in feet.

- Precipitation is the mean daily precipitation data provided by the IMC, based on the inputs into the SFWMM. Precipitation inputs were for the 10 cells that encompass the EAA Reservoir A-1 footprint. Inflow data was based on actual precipitation values for the POS. The average value of all 10 cells for each day in the POS was used as input data for the WBM. The average yearly precipitation over the POS is approximately 51 inches, with a maximum of 68.0 inches in water year 1970 and a minimum of 40.4 inches in water year 1971.

6.2.5 Outflow

EAA Reservoir A-1 outflows in the WBM consist of evaporation, seepage, environmental deliveries, agricultural deliveries, and excess volume outflows. A description of each outflow is provided below.

- Evaporation – Mean daily evapotranspiration (ET) data (for the POS) for the 10 cells that encompass the EAA Reservoir A-1 footprint were provided by the IMC, based on the inputs into the SFWMM. The ET data used in the SFWMM were compared to historical direct evaporation data. Historical evaporation data were downloaded from the SFWMD's hydrometeorologic, water quality, and hydrogeologic data retrieval system known as DBHYDRO for the area in the vicinity of the EAA Reservoir A-1. The data provided by DBHYDRO is pan evaporation. A commonly accepted conversion of pan evaporation to actual evaporation is 70 percent of the pan evaporation equals actual evaporation. Using this conversion, a comparison of the ET data used in the SFWMM to actual evaporation data revealed little difference between the two values. As a result, the average value of the ET data from all 10 cells was used as the evaporation data for the WBM. The average yearly ET over the POS is approximately 44 inches, with a maximum value of 49.2 inches in water year 1967 and a minimum value of 36.7 inches in water year 1974.
- Seepage – Total seepage from the EAA Reservoir A-1, as estimated by the seepage analysis work performed by Black & Veatch during the Test Cells Program. The total seepage varies with the EAA Reservoir A-1 water depth and depends on the seepage reduction alternative selected. For a scenario with an embankment that includes a 30 foot deep seepage cutoff wall and a 20 foot deep seepage canal, the total seepage may be approximated with the linear equation:

$$\text{Total Seepage} = 25.951x$$

Where:

Total Seepage is the total seepage from the EAA Reservoir A-1 Project in cfs.

x is the EAA Reservoir A-1 Project water depth in feet.

- Environmental Deliveries – Environmental deliveries required from the EAA Reservoir A-1 Project to meet a specific environmental allocation in the Everglades via STA-3/4. Environmental deliveries data were provided by the OoM, based on the ECP 2015 simulation. The environmental deliveries are equal to:

$$\text{Environmental Deliveries} = WCS4W + WCS4S + EVBLSW + EVBLSS + FLIMPM + FLIMPN + EARA2O$$

- Agricultural Deliveries – Specific agricultural deliveries in the EAA to be supplied by the EAA Reservoir A-1 Project. Agricultural deliveries data were provided by the OoM, based on the ECP 2010 simulation. The agricultural deliveries are equal to:

$$\text{Agricultural Deliveries} = SDMDLKMIA + EARMA1 + EARMA2 + SDMDLKNNRH + EARNH1 + EARNH2$$

For the water balance analysis, it was assumed that the EAA Reservoir A-1 Project would supply the environmental deliveries with the storage available before supplying the agricultural deliveries, and after accounting for evaporation and seepage losses. In addition, it was assumed that during EAA Reservoir A-1 stages at or below the minimum WSE of 0.5 ft., the available flows in the canals would be used to increase the EAA Reservoir A-1 water level over the minimum WSE, before supplying any of the deliveries.

- Excess Volume Outflows – Flows discharged from the EAA Reservoir A-1 Project when full and inflows are greater than outflows. These flows are released to maintain the maximum WSE of the EAA Reservoir A-1.

6.2.6 EAA Reservoir A-1 Performance, Sizing and Yield

The EAA Reservoir A-1 provides a storage capacity of 190,551 acre-feet at a water depth of 12 feet. The EAA Reservoir A-1 footprint is 15,833 acres. As mentioned in the previous section, the EAA Reservoir A-1 includes a 26-foot tall embankment and 3H:1V side slopes.

Releases from the EAA Reservoir A-1 include environmental deliveries via STA-3/4 and agricultural deliveries for the NNRC/Hillsboro Canal basin. The ECP 2010 and ECP 2015 version 5.4.2 runs simulate the amount of flow required from the EAA Reservoir A-1 on a daily basis to supply the environmental deliveries and the agricultural deliveries, respectively.

To assess the performance of the EAA Reservoir A-1, it was necessary to determine how the EAA Reservoir A-1 would operate during a typical dry, average, and wet water year. An assessment was made using the available rainfall data from 1 January 1965 to 31 December 2000 to determine the typical dry, average, and wet water year for the EAA Reservoir A-1. The data used were the daily average rainfall over the 10 model cells that simulate the EAA Reservoir A-1 footprint, in the SFWMM. The assessment was based on water years, which run from October 1st to September 30th (i.e. water year 1970 runs from October 1, 1969 to October 30, 1970). The available data cover a period of 36 water years.

For each of these water years the sum of the daily rainfall was calculated to give the yearly rainfall. A graph showing the distribution of the yearly rainfall is provided in Figure 6.2-2. Based on this analysis, the mean yearly rainfall and the standard deviation of the yearly rainfall were calculated. Each water year was then categorized as “Wet,” “Average” or “Dry” based on the following assumptions:

- A wet year is more than one standard deviation above the mean rainfall
- An average year is within one standard deviation of the mean rainfall
- A dry year is more than one standard deviation below the mean rainfall

Figure 6.2-2 Distribution of Yearly Rainfall

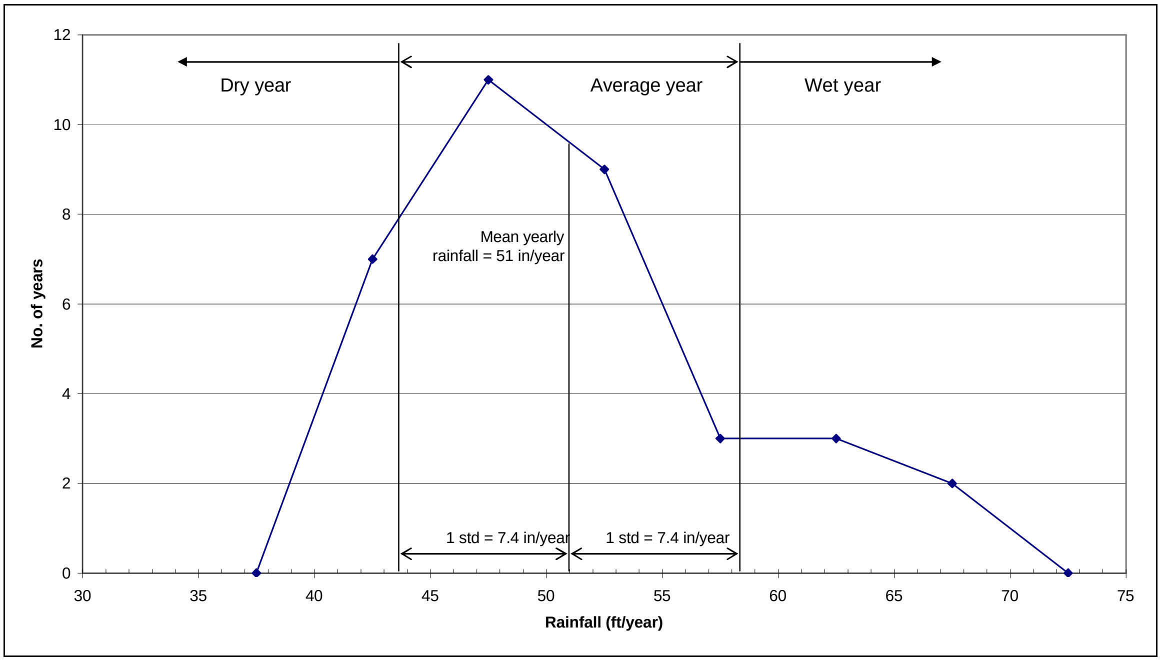


Table 6.2-3 summarizes how the data is distributed between dry, average and wet years based on these assumptions.

Table 6.2-3 Summary of Yearly Rainfall Analysis

Category	Total Number of Years
Wet year	6
Average year	25
Dry year	4

It was then necessary to choose one year in each category to best represent the historical data. The final choice of year was engineering judgment, based on the investigation of several properties of each of the years. These were as follows:

- Rainfall data was summed over 2-year, 5-year, and 10-year periods to identify rainfall trends that extended beyond one year. It was generally found that even during a dry year the quantity of rainfall during the wet season was sufficient to refill the EAA Reservoir A-1. Therefore, weather trends of greater duration will have negligible effect on operation of the EAA Reservoir A-1. With this in mind, it was valid to consider each year in isolation.
- A summary of the number of years during which environmental and agricultural deliveries were met was performed. The analysis was based on the WBM EAA-A1_2015_Envtl.xls. Table 6.2-4 summarizes the results of this analysis.

Table 6.2-4 Summary of Demands Met with Years Categorized as Wet, Dry or Average

			Agricultural Deliveries Not Met		Environmental Deliveries Not Met		Environmental Deliveries Met, Agricultural Deliveries Not Met	
Year	**No. of Years**	**No. of Days**	**Days**	**Percentage of Days**	**Days**	**Percentage of Days**	**Days**	**Percentage of Days**
All Dry Years	4	1460	201	14	161	11	3	0
1971	1	365	110	30	105	29	0	0
All Average Years	25	9125	1171	13	1448	16	45	0
1992	1	366	41	11	65	18	2	1
All Wet Years	6	2190	141	6	300	14	5	0
1978	1	365	0	0	1	0	0	0

The "Dry" water year would have the greatest percentage of deliveries not met among the other dry years. The "Average" water year would have the percentage of deliveries met close to the values for the other average years. The "Wet" water year would have the smallest percentage of deliveries not met among the other wet years. Based on this assessment, the following years were chosen to best represent the general trends during a "Dry," "Average," and a "Wet" water year:

- Dry year = 1971
- Average year = 1992
- Wet year = 1978

Based on the ECP 2015 run, the average annual environmental delivery supplied by the EAA Reservoir A-1 via STA-3/4, as determined for the deliveries by the WBM, is approximately 685,000 acre-feet, with a maximum of 1,487,000 acre-feet in water year 1983 and a minimum of 104,000 acre-feet in water year 1990. The current average annual inflow into STA-3/4 is approximately 656,000 acre-feet (Piccone, 2005). The total deliveries over the POS are approximately 24,213,000 acre-feet.

Based on the ECP 2010 run, the average annual agricultural delivery supplied by the EAA Reservoir A-1, as determined for the deliveries by the WBM, is 84,000 acre-feet, with a maximum of 160,000 acre-feet in water year 1985 and a minimum of 19,000 acre-feet in water year 1970. The total deliveries over the POS are approximately 3,073,000 acre-feet.

The EAA Reservoir A-1 yield is defined as the sum of the environmental and agricultural deliveries supplied by the EAA Reservoir A-1. The average annual yield of the EAA Reservoir A-1 is approximately 769,000 acre-feet, with a maximum of 1,538,000 acre-feet in water year 1983 and a minimum of 207,000 acre-feet in water year 1971. The total yield over the POS is approximately 26,900,000 acre-feet.

Figure 6.2-3 illustrates the average annual inflows and outflows of the EAA Reservoir A-1. In addition, Figures 6.2-4 through 6.2-6 illustrate the EAA Reservoir A-1 operations for the selected “Average” and “Wet” water years, and for the “Dry” period of water years 1971 to 1972. The performance of the EAA Reservoir A-1 for the selected “Dry”, “Average”, and “Wet” water years is provided in Table 6.2-5 and the mass balance for each of the selected years is provided in Table 6.2-6.

Table 6.2-5 Summary of Deliveries Met With Years Categorized as Dry, Average, or Wet

		1971 "Dry" Year	1992 "Average" Year	1978 "Wet" Year	Complete POS
EAA Reservoir A-1 Inflows	NNRC inflow, acre-feet	179,165	470,155	346,118	12,906,675
	Miami Canal inflow, acre-feet	227,050	529,297	233,841	13,229,975
	Precipitation, acre-feet	53,443	71,879	78,144	2,423,429
EAA Reservoir A-1 Outflows	Environmental deliveries, acre-feet	123,023	857,780	624,402	22,518,200
	Agricultural deliveries, acre-feet	74,451	103,491	80,440	3,073,453
	Evaporation, acre-feet	59,730	62,708	52,156	2,081,752
	Seepage, acre-feet	22,687	10,913	30,567	639,218
	Excess volume outflows, acre-feet	12,599	0	0	185,494
EAA Reservoir A-1 Volumes	Start of year, acre-feet	7,827	7,827	188,775	0
	End of year, acre-feet	174,995	44,266	59,313	62,451
Deliveries	Environmental deliveries, acre-feet	360,492	1,243,166	772,530	31,778,063
	Environmental deliveries supplied by canals, acre-feet	9,438	71,506	148,079	1,694,324
	Percentage of environmental deliveries met	37 percent	75 percent	100 percent	76 percent
	Agricultural deliveries, acre-feet	176,933	126,313	80,440	4,755,705
	Percentage of agricultural deliveries met	42 percent	82 percent	100 percent	65 percent

-During a Wet year, less environmental and agricultural deliveries are required from the EAA Reservoir A-1 (not including the environmental deliveries supplied by canals), resulting in higher water levels in the reservoir. Therefore, less canal inflows go into the reservoir and seepage from the reservoir increases.

-Total environmental deliveries supplied by the system include environmental deliveries from the EAA Reservoir A-1 and environmental deliveries supplied by canals.

-Dry, Average, and Wet years are based on the rainfall analysis discussed in Section 6.2.6, and not the available flows, environmental, and agricultural deliveries simulated in the ECP runs.

-EAA Reservoir A-1 volume at the minimum water depth of 0.5 feet is 7,827 acre-feet.

-Average annual values are illustrated in Figure 6.2-3.

Figure 6.2-3 Average Annual Inflows and Outflows of the EAA Reservoir A-1

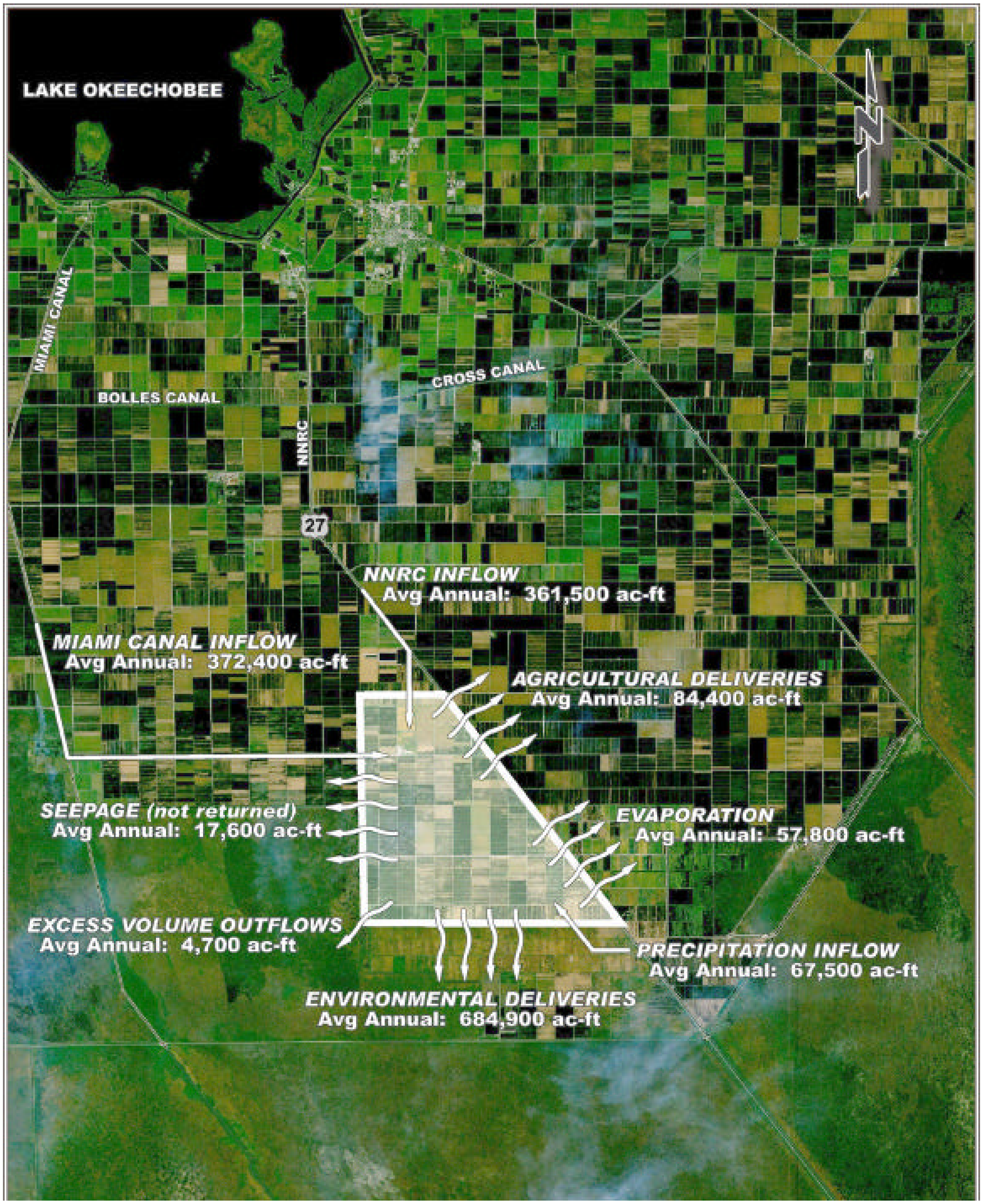


Figure 6.2-4 EAA Reservoir Operation during an "Average" Water Year

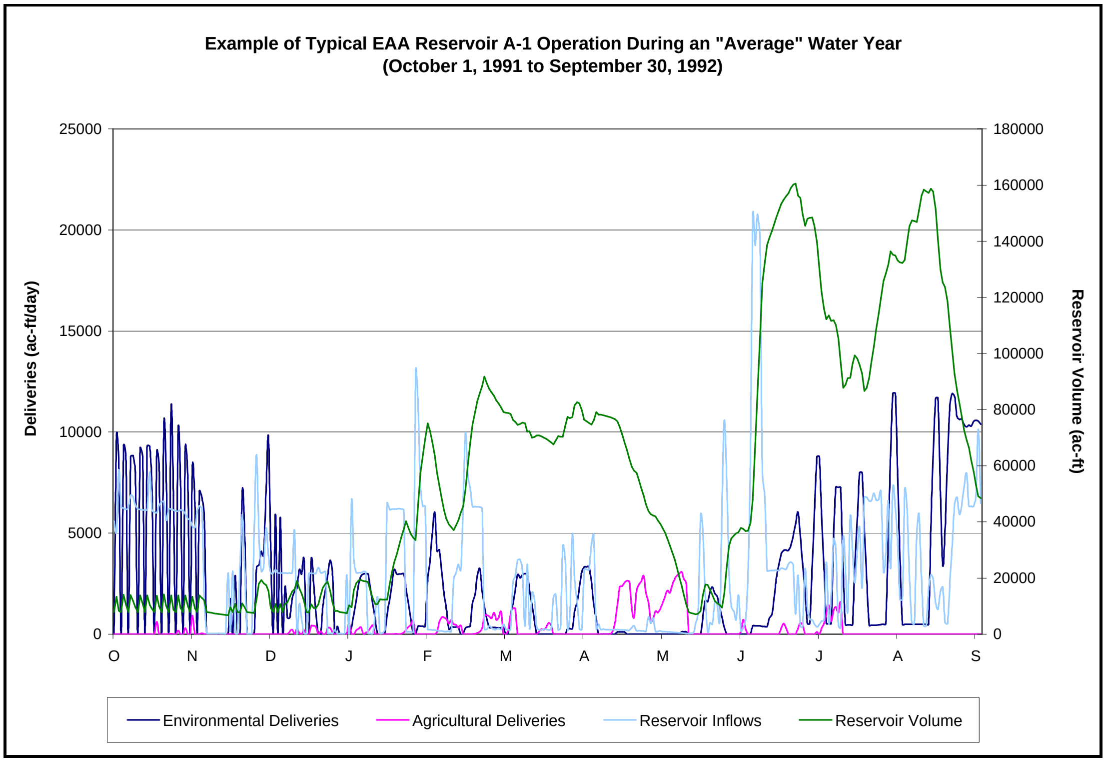


Figure 6.2-5 EAA Reservoir Operation during a "Wet" Water Year

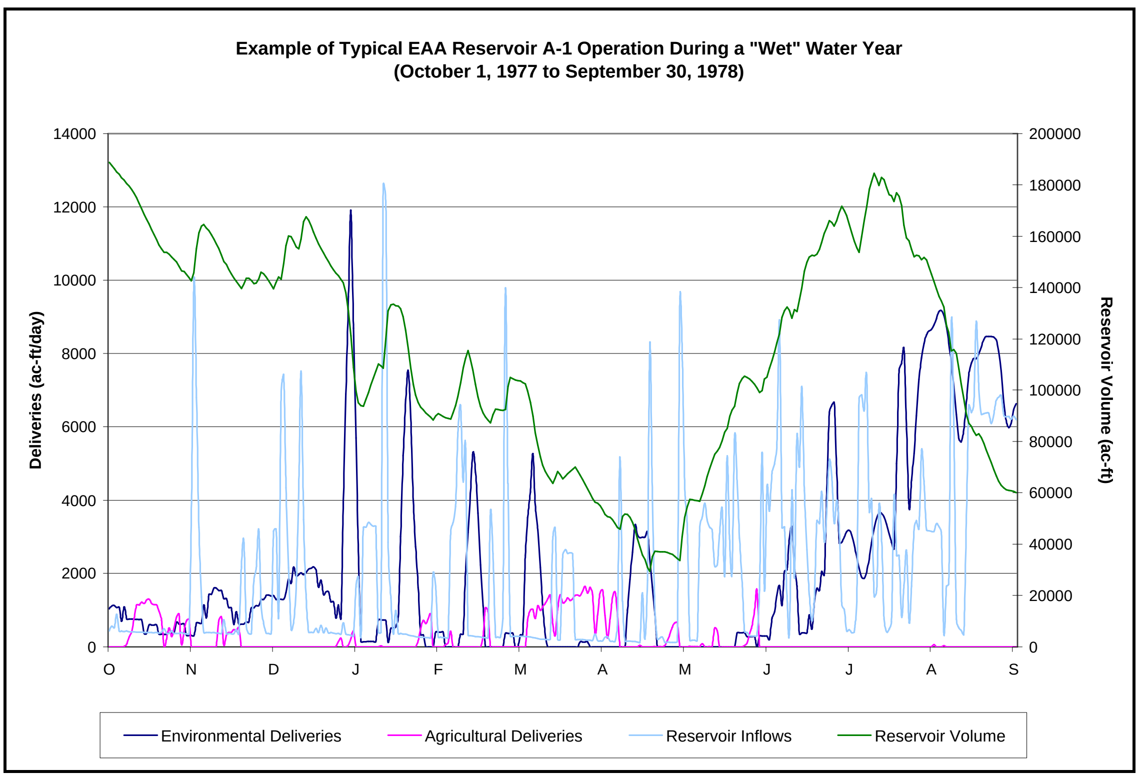


Figure 6.2-6 EAA Reservoir Operation during the "Dry" Water Year

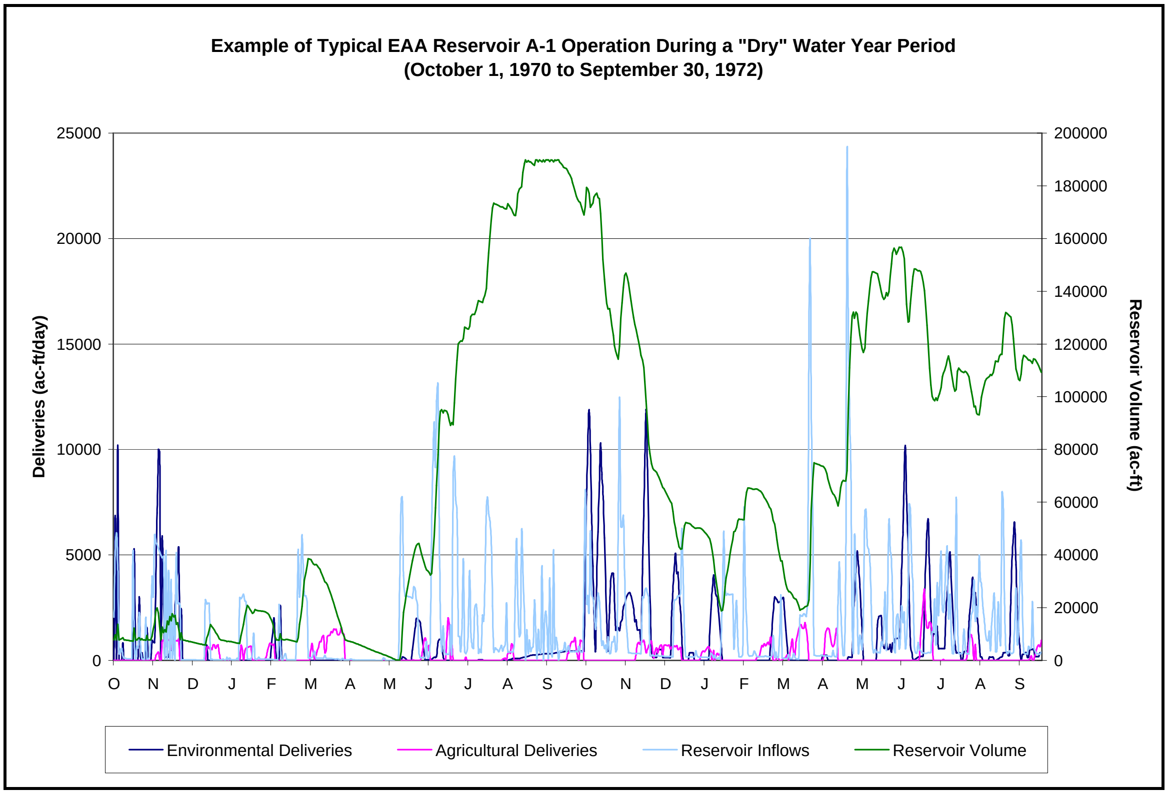


Table 6.2-6 Mass Balance for Selected Water Years

	1971 "Dry" Year	1992 "Average" Year	1978 "Wet" Year	Complete POS
Initial EAA Reservoir A-1 volume, acre-feet	7,827	7,827	188,775	0
Total EAA Reservoir A-1 inflow, acre-feet	459,658	1,071,331	658,103	28,560,079
Total volume in, acre-feet	467,486	1,079,159	846,878	28,560,079
Total outflow, acre-feet	292,491	1,034,892	787,565	28,498,116
Final EAA Reservoir A-1 volume, acre-feet	174,995	44,266	59,313	62,451
Total volume out, acre-feet	467,485	1,079,159	846,879	28,560,567

-During a Wet year, less environmental and agricultural deliveries are required from the EAA Reservoir A-1 (not including the environmental deliveries supplied by canals), resulting in higher water levels in the reservoir. Therefore, less canal inflows go into the reservoir and seepage from the reservoir increases.

-EAA Reservoir A-1 volume at the minimum water depth of 0.5 feet is 7,827 acre-feet.

The WBM also shows that the EAA Reservoir A-1 is full 133 days over the POS. The EAA Reservoir A-1 is able to supply 76.2 percent of the environmental deliveries by volume from the

ECP 2015 simulation and 64.6 percent of the agricultural deliveries by volume from the ECP 2010 simulation. These values assume the following EAA Reservoir A-1 conditions:

- The EAA Reservoir A-1 starts empty and attempts to meet 100 percent of the environmental deliveries via STA-3/4 and only agricultural deliveries for the NNRC/Hillsboro Canal basin.
- Attempt to capture100 percent of the available flows in the NNRC, Miami Canal, and seepage canals for inflow into the EAA Reservoir A-1.
- An EAA Reservoir A-1 minimum depth of 0.5 foot, below which flows to STA-3/4 and agricultural deliveries cannot be supplied
- Northeast pump station pumps to 12 feet of EAA Reservoir A-1 depth at a rate of 3,600 cfs.
- Pump stations G-370 and G-372 are not modified and pump to 8 feet of EAA Reservoir A-1 water depth at a rate of up to 2,340 cfs and 3,120 cfs, respectively
- EAA Reservoir A-1 outflows to STA-3/4 are capped at 6,000 cfs, the rated capacity of the STA-3/4 inflow structures.

The results of the WBM run are shown on Figures 6.2-7 through 6.2-16.

Figure 6.2-7 Water Balance Model

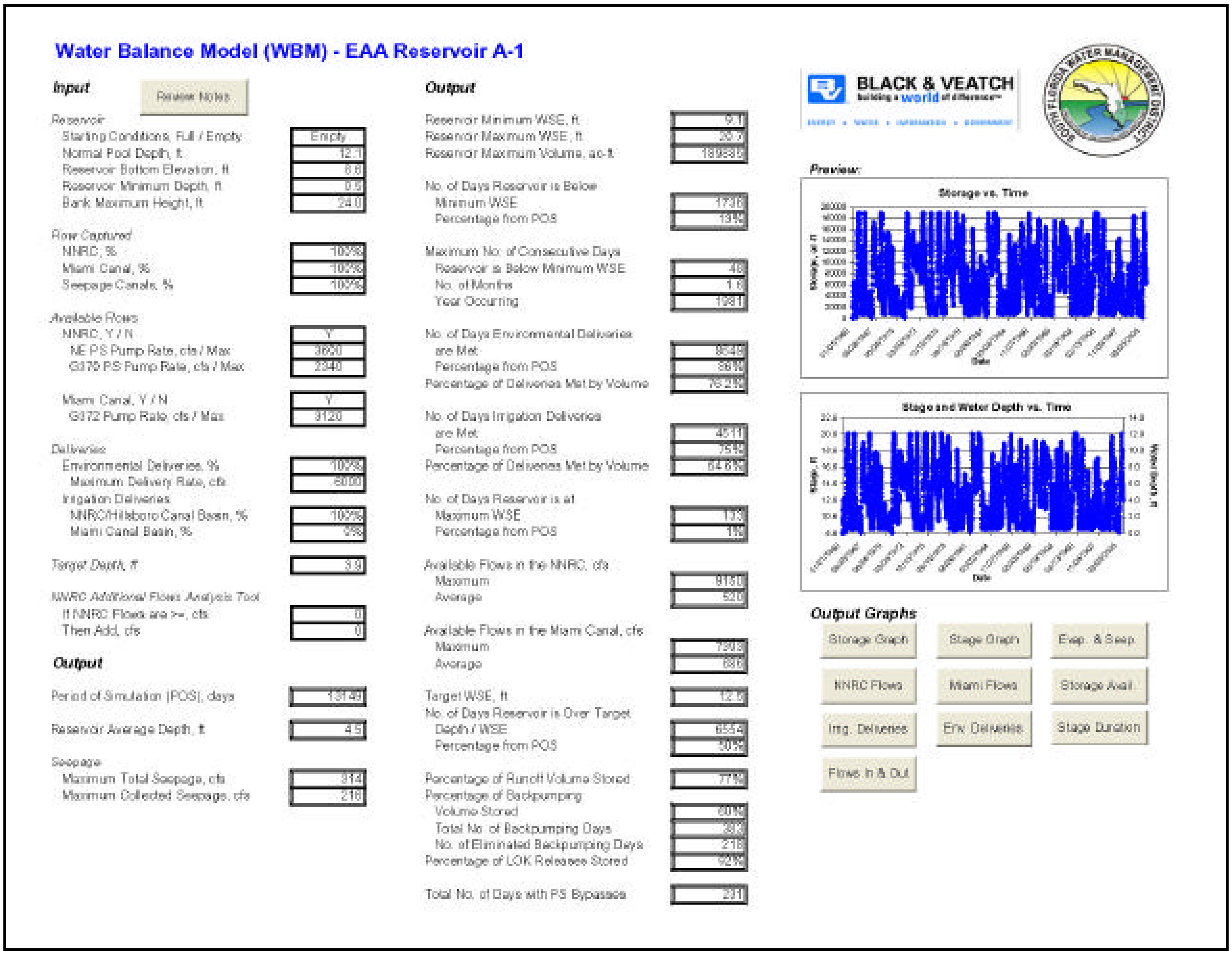
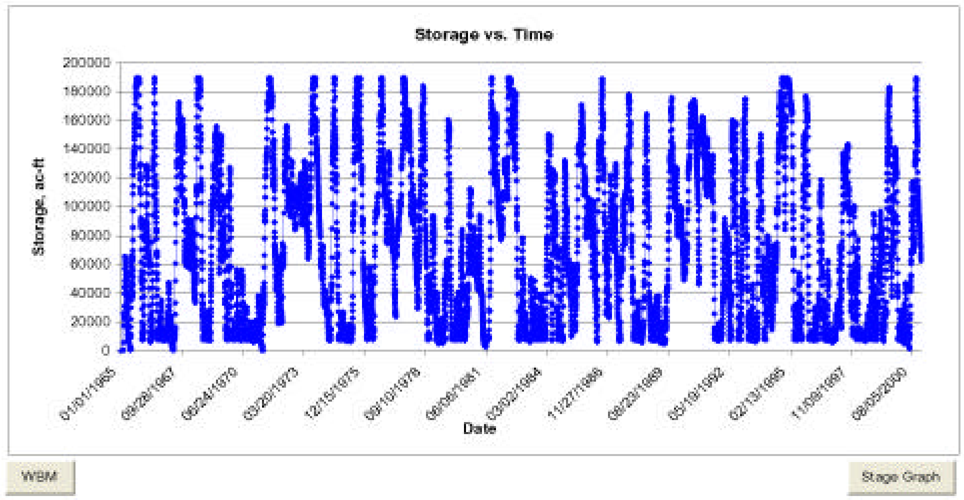
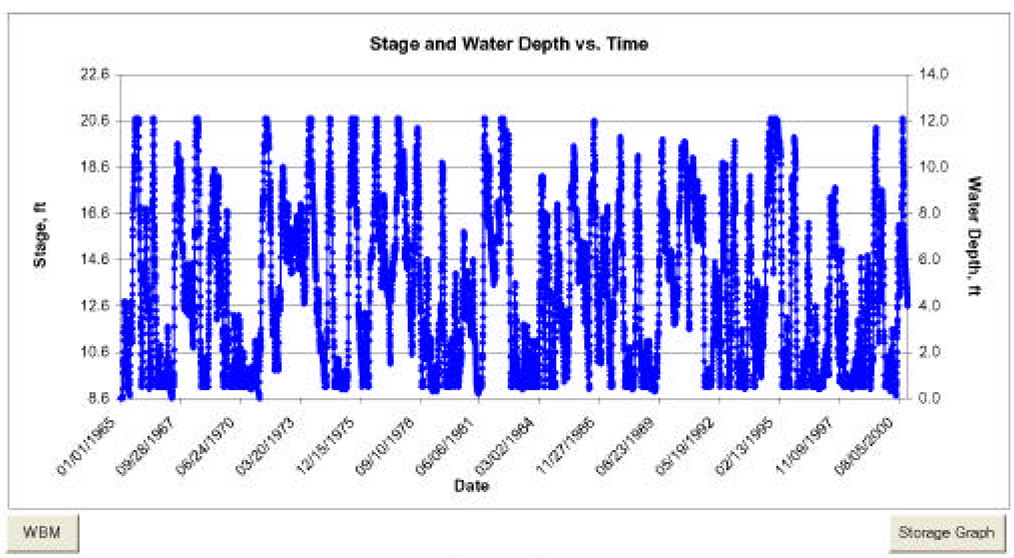


Figure 6.2-8 Storage vs. Time



Summary Input:		*Summary Output:*	
Starting Conditions, Full / Empty	Empty	Reservoir Maximum Volume, ac-ft	189885
Normal Pool Depth, ft	12.1	Reservoir Minimum WSE, ft	9.1
Reservoir Bottom Elevation, ft	8.6	Percentage from POS Reservoir is Below Minimum WSE	13%
Reservoir Minimum Depth, ft	0.5	Maximum No. of Consecutive Days Reservoir is below	
NNRC, Y / N	Y	Minimum WSE	48
Miami Canal, Y / N	Y	No. of Months	1.6
Environmental Deliveries, %	100%	Year Occurring	1981
Irrigation Deliveries		Percentage of Environmental Deliveries Met by Volume	76%

Figure 6.2-9 Stage and Water Depth vs. Time



Summary Input:		*Summary Output:*	
Starting Conditions, Full / Empty	Empty	Reservoir Maximum WSE, ft	20.70
Normal Pool Depth, ft	12.1	Reservoir Minimum WSE, ft	9.1
Reservoir Bottom Elevation, ft	8.6	Percentage from POS Reservoir is Below Minimum WSE	13%
Reservoir Minimum Depth, ft	0.5	Maximum No. of Consecutive Days Reservoir is below	
NNRC, Y / N	Y	Minimum WSE	48
Miami Canal, Y / N	Y	No. of Months	1.6
Environmental Deliveries, %	100%	Year Occurring	1981
Irrigation Deliveries		Percentage of Environmental Deliveries Met by Volume	76%
NNRC/Hillsboro Canal Basin, %	100%	Percentage of Irrigation Deliveries Met by Volume	65%

Figure 6.2-10 NNRC Flows vs. Time

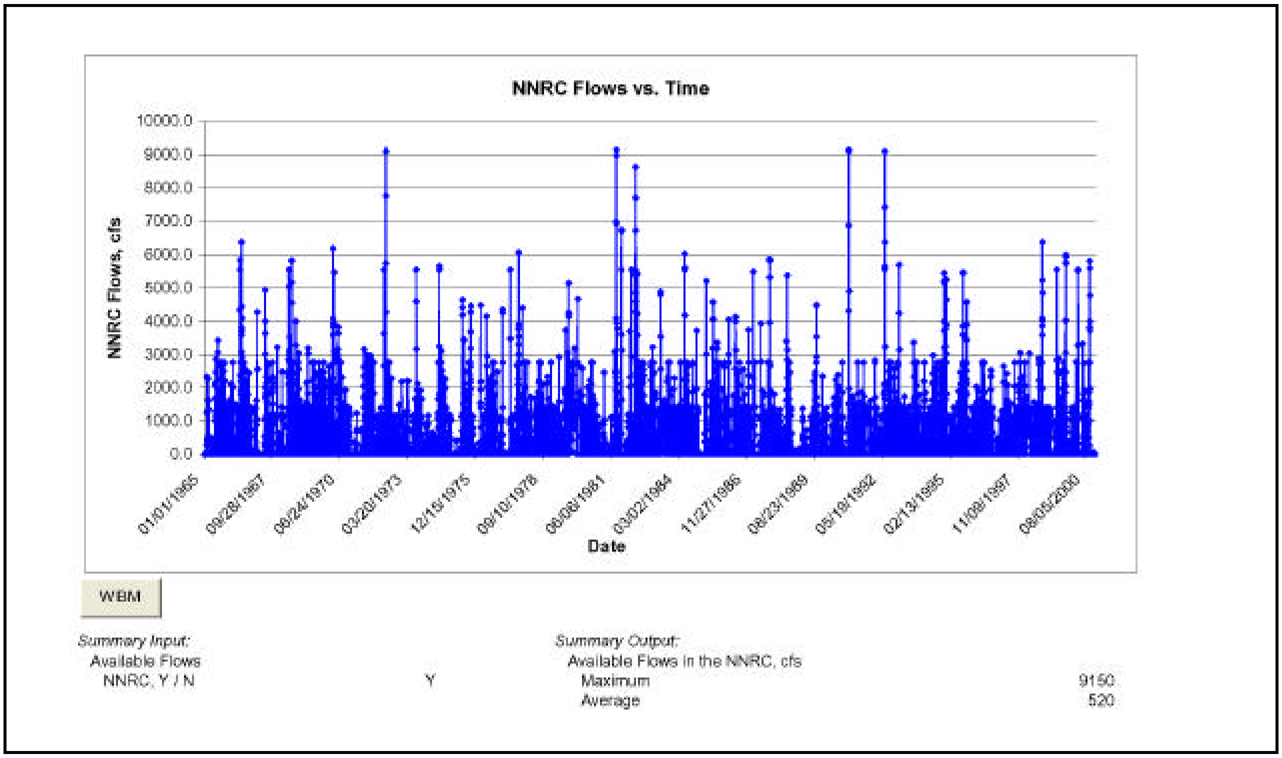


Figure 6.2-11 Miami Canal Flows vs. Time

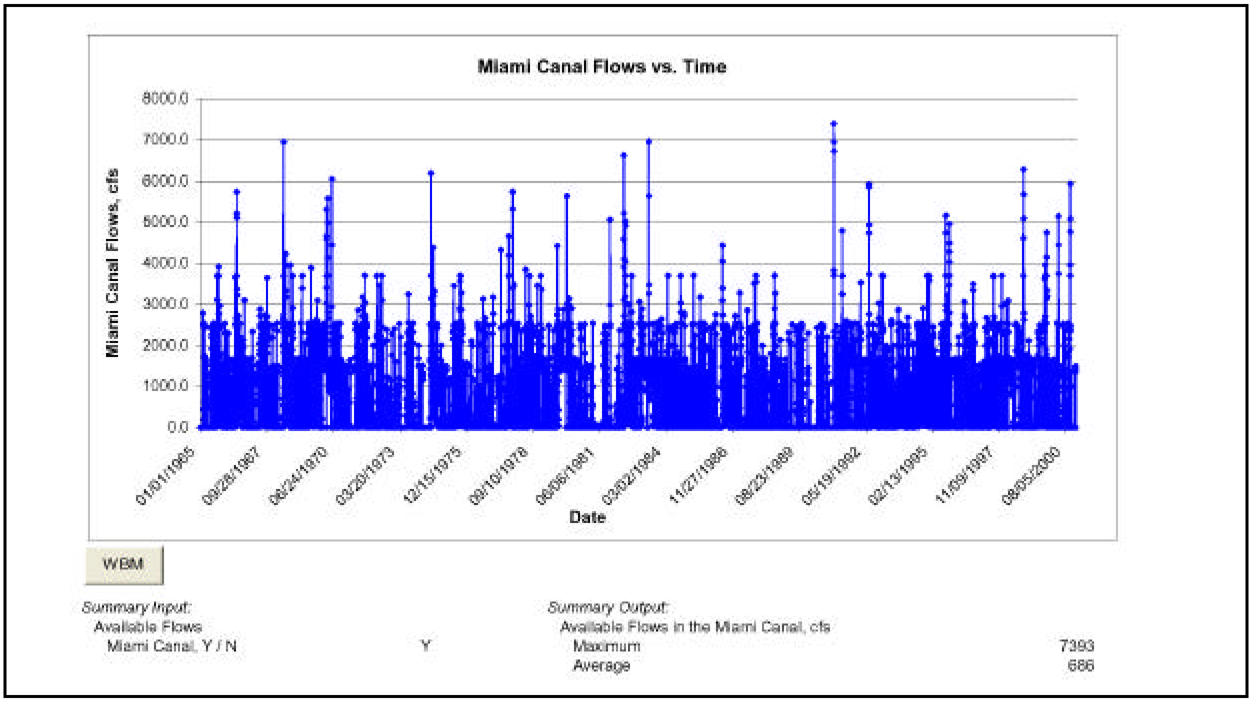
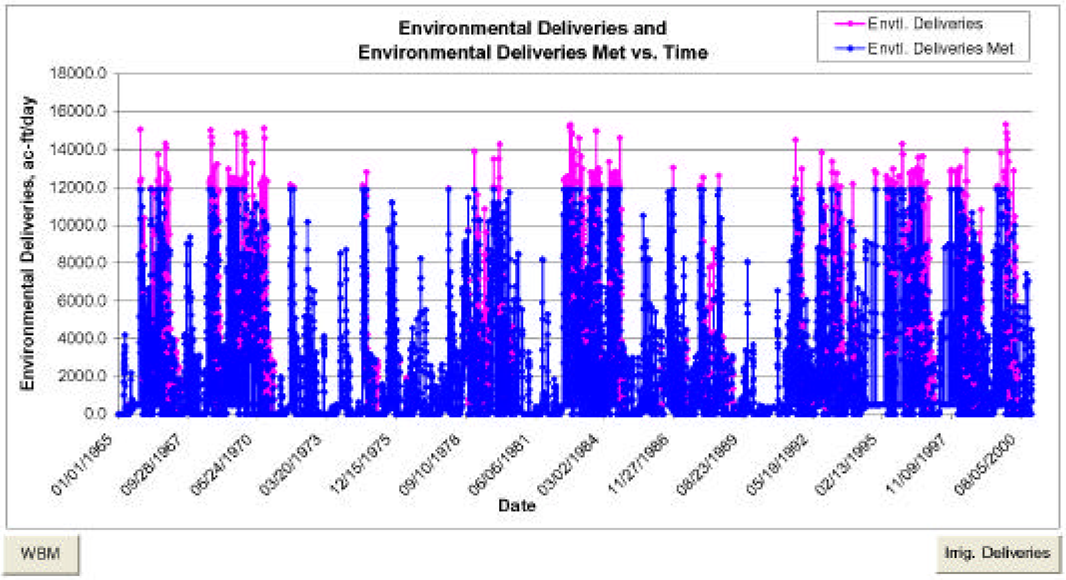
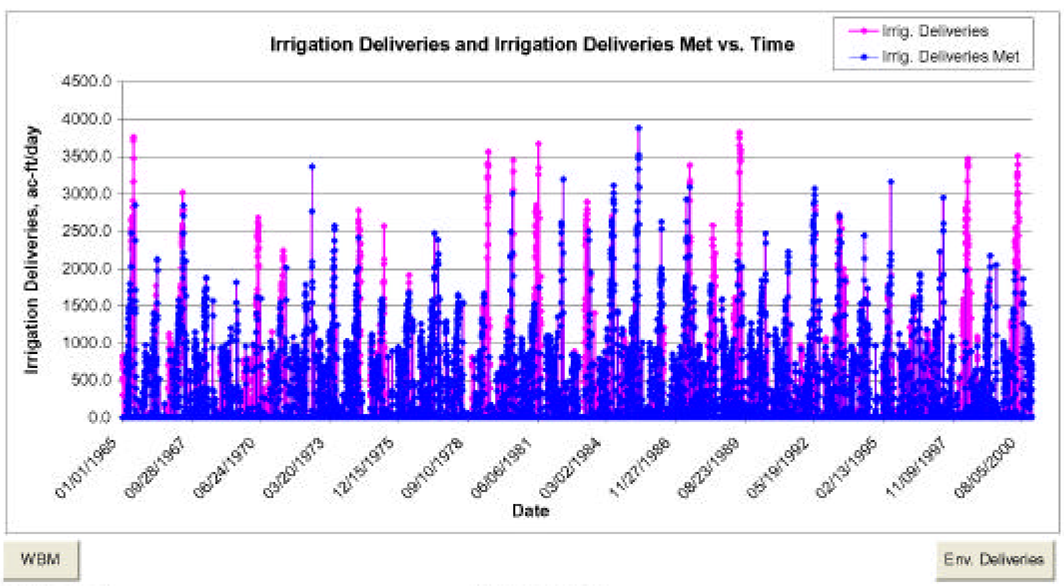


Figure 6.2-12 Environmental Deliveries and Environmental Deliveries Met vs. Time



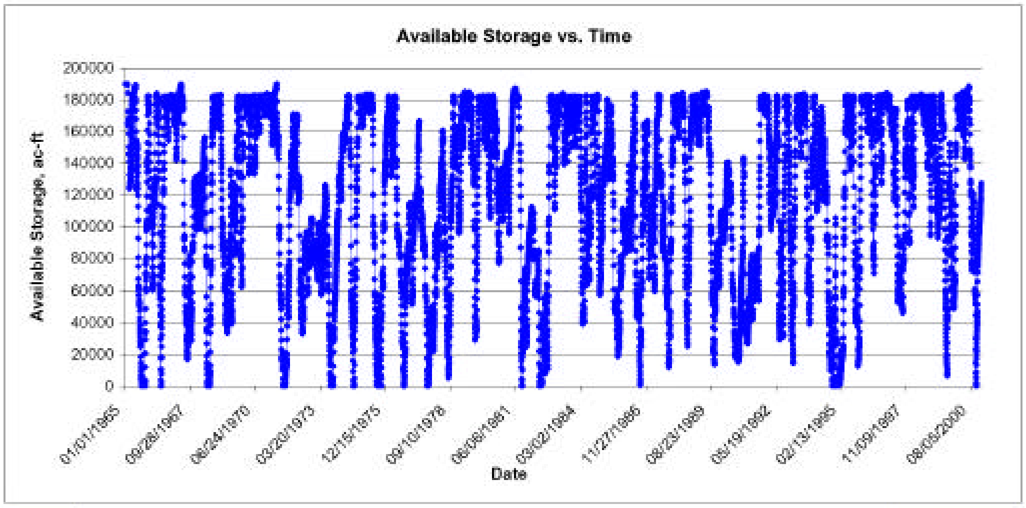
Summary Input:		*Summary Output:*	
Environmental Deliveries	100%	No. of Days Environmental Deliveries are Met	8649
		Percentage from POS	86%
		Percentage of Deliveries Met by Volume	76%

Figure 6.2-13 Agricultural Deliveries and Agricultural Deliveries Met vs. Time



Summary Input:		*Summary Output:*	
Irrigation Deliveries		No. of Days Irrigation Deliveries are Met	4511
NNRC/Hillsboro Canal Basin, %	100%	Percentage from POS	75%
Miami Canal Basin, %	0%	Percentage of Deliveries Met by Volume	65%

Figure 6.2-14 Available Storage vs. Time



WBM

Storage Graph

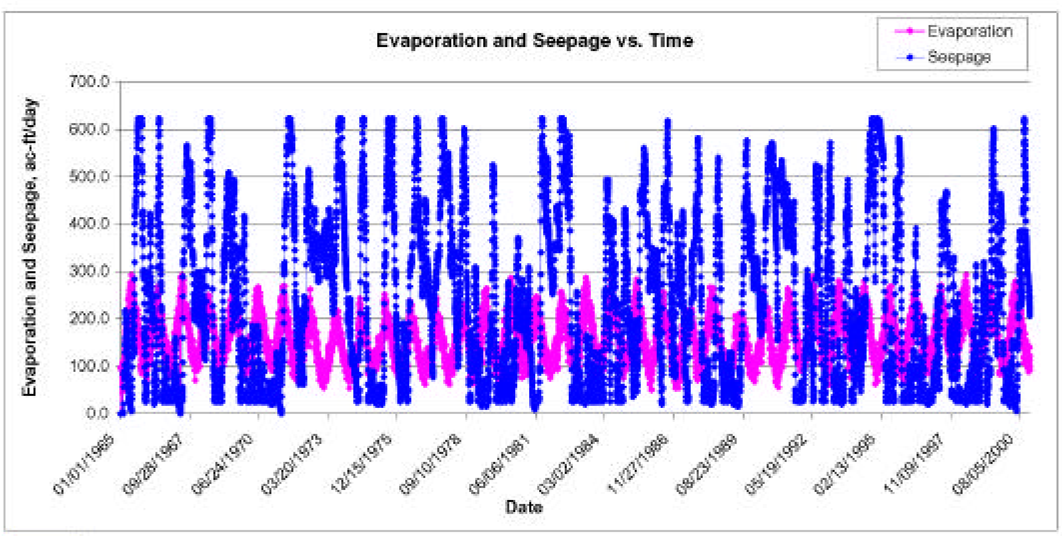
Summary Input:

Starting Conditions, Full / Empty	Empty
Normal Pool Depth, ft	12.1
Reservoir Bottom Elevation, ft	8.6
Reservoir Minimum Depth, ft	0.5
NNRC, Y / N	Y
Environmental Deliveries, %	100%
Irrigation Deliveries	
NNRC/Hillsboro Canal Basin, %	100%

Summary Output:

Reservoir Maximum Volume, ac-ft	189885
Reservoir Minimum WSE, ft	9.1
Percentage from POS Reservoir is Below Minimum WSE	13%
Maximum No. of Consecutive Days Reservoir is below	
Minimum WSE	48
No. of Months	1.6
Year Occurring	1981
Percentage of Environmental Deliveries Met by Volume	76%

Figure 6.2-15 Evaporation and Storage vs. Time

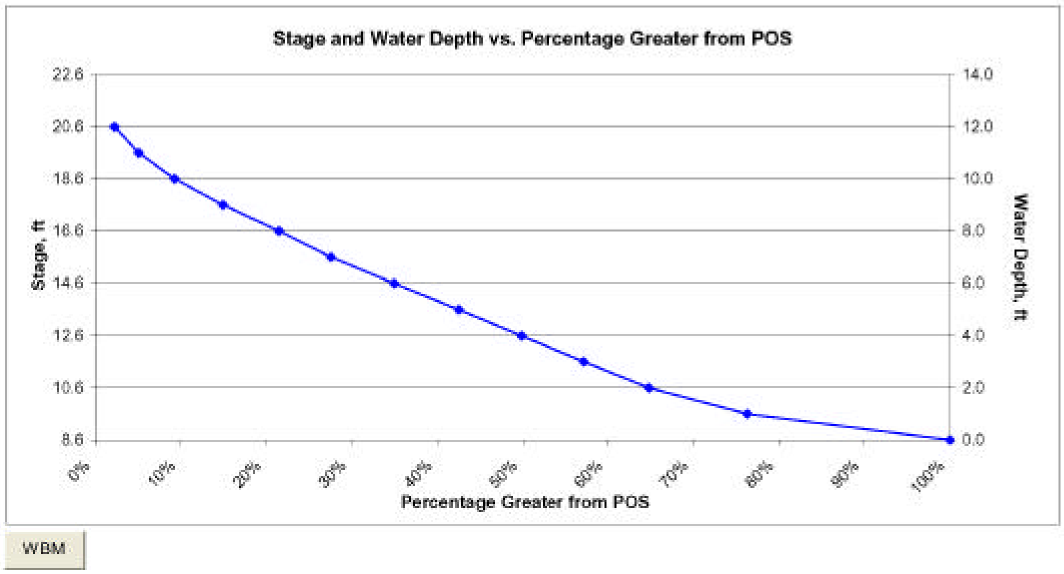


WBM

Summary Output:

Average Evaporation, ac-ft	158
Average Seepage from Reservoir, ac-ft	233
Maximum Total Seepage, cfs	314
Maximum Total Seepage ac-ft/day	623
Maximum Collected Seepage, cfs	216
Maximum Collected Seepage ac-ft/day	429

Figure 6.2-16 Stage and Water Depth vs. Percentage Greater from POS



6.3 ENVIRONMENTAL AND AGRICULTURAL DELIVERIES

The environmental calls and the specific agricultural calls via STA-3/4 to be supplied by the EAA Reservoir A-1 were provided by the OoM, based on the ECP 2010 and ECP 2015 version 5.4.2. simulations. The calls simulated by OoM are different than the deliveries determined by the WBM.

Environmental deliveries are the flows from the EAA Reservoir A-1 via STA-3/4 intended to meet a specific environmental delivery in the Everglades. Based on the ECP 2015 simulation, the environmental deliveries are equal to:

Environmental Deliveries = WCS4W + WCS4S + EVBLSW + EVBLSS + FLIMPM + FLIMPN + EARA2O

Agricultural deliveries are the specific agricultural deliveries in the EAA to be supplied by the EAA Reservoir A-1. The agriculture deliveries are equal to:

Agricultural Deliveries = SDMDLKMIA + EARMA1 + EARMA2 + SDMDLKNNRH + EARNH1 + EARNH2

The results of the ECP 2015 simulation from OoM indicate that the average annual environmental calls from the EAA Reservoir A-1 via STA-3/4 is approximately 901,000 acre-feet, with a maximum of 2,256,000 acre-feet in water year 1970 and a minimum of 104,000 acre-feet in water year 1990. The current average annual inflow into STA-3/4 is approximately 656,000 acre-feet (Piccone, 2005). The total required delivery over the POS is approximately 31,800,000 acre-feet.

The results of the ECP 2010 simulation indicate that the average annual agricultural calls from the EAA Reservoir A-1 to the NNRC/Hillsboro Canal basin is approximately 129,000 acre-feet, with a maximum of 215,000 acre-feet in water year 2000 and a minimum of 77,000 acre-feet in water year 1969. The total required delivery over the POS is approximately 4,800,000 acre-feet.

6.4 WATER QUALITY

6.4.1 Model Description

The DMSTA was developed by Dr. Bill Walker and Dr. Bob Kadlec under contract with the DOI and the USACE to support the design of wetlands in STAs, which are capable of removing phosphorus from stormwater runoff from the EAA and Lake Okeechobee releases. DMSTA simulates daily water and mass balances in a user-defined series of wetland treatment cells (Continuous Stirred Tank Reactors), each with specified morphometry, hydraulics, and phosphorus cycling parameters.

Water-balance terms for each cell include inflow, bypass, rainfall, ET, outflow, and seepage in and out of a cell. Parameter estimates for the phosphorus cycling model were developed for various vegetation types. The model is coded in visual basic for applications; the user interface is a Microsoft Excel workbook.

Compared with typical marsh treatment areas in the STAs, CERP storage EAA reservoir designs, such as the EAA Reservoir A-1, tend to have greater mean depths, greater variations in depth, and longer water residence times. These factors can be expected to have significant effects on vegetation communities, phosphorus dynamics, and model calibrations. Currently, STAs are operated at a static water depth of 1.2 to 1.5 feet. Deteriorations in vegetation integrity and performance have been observed in cells with prolonged water depths exceeding 2.5 to 3 feet. Current designs for CERP storage EAA reservoirs have maximum depths ranging from 6 to 12 feet. The expected maximum operating depth for the EAA Reservoir A-1 is 12 feet. The EAA Reservoir A-1 embankment will be designed to store the PMP and to accommodate wave run-up above the 12-foot operating depth.

6.4.2 DMSTA2 Results

6.4.2.1 Phosphorus

To predict phosphorus levels in the EAA Reservoir A-1 with DMSTA2, time series of inflows and outflows to the EAA Reservoir A-1 were imported from an OoM ECP 2010 model for the simulation period 1965 to 2000. Inflows included available flows from the NNRC at G-370 and NE pump stations; available flows from the Miami Canal at G-372 pump station; and historic rainfall data corresponding to each year of the simulation period.

Outflows included releases to environmental and agricultural areas and simulated ET, corresponding to each year of the simulation period, seepage, and any discharges when the water level exceeds the expected operating depth of 12 feet.

The 1965 to 2000 time series of phosphorus concentrations associated with the NNRC and the Miami Canal was based on monthly average concentrations developed by Burns & McDonnell as part of the Regional Feasibility Study (under contract to ADA Engineering, Inc.).

The times series described above were imported to DMSTA2, which produced continuous daily simulations of water and phosphorus mass balances over the long-term simulation period.

DMSTA2 predicted that for the period of simulation, 76 percent of the total phosphorus loading came from the two canals, while only three percent and 21 percent were from rainfall and recycled seepage, respectively. Of the total phosphorus load leaving the EAA Reservoir A-1, 32 percent and 27 percent was released to meet environmental and agricultural deliveries, respectively. Another 32 percent of the total load release was from seepage, although 75 percent of the seepage released was recycled back to the EAA Reservoir A-1. The flow weighted average concentration of phosphorus in the releases was 68 parts per billion (ppb) during the simulation period compared to an average of 82 ppb associated with the inflows.

The difference between the phosphorus inflow loading and outflow loading is the amount of phosphorus deposited in the EAA Reservoir A-1 sediments. If the amount of deposited sediments is subtracted from the average annual total loading from the canals, the EAA Reservoir A-1 is estimated to achieve an average 17 percent reduction in the phosphorus loading from the canals.

Figure 6.4-1 is a summary of the phosphorus mass balance for the simulation period. The Table also includes predicted mass balances for 1983, one of the years of highest phosphorus loading, and for 1975, one of the years of lowest phosphorus loading. It should be noted that since the EAA Reservoir A-1 does not currently exist, the previously described results are predictions of phosphorus if the EAA Reservoir A-1 had been in place, assuming historic meteorological conditions and the future operational strategy incorporated in the ECP 2010 model.

Table 6.4-1 Phosphorus Mass Balance

Inflow	**1983** **10^3 kg P per year**	**1975** **10^3 kg P per year**	**1965-2000** **10^3 kg P per year**
Canals	98.2	21.2	49.6
Rainfall	2.1	2.1	2.1
Seepage Recycle	16.3	11.2	13.5
Total	116.6	34.5	65.2
Outflow			
Agricultural deliveries	20.3	8.8	15.1
Environmental deliveries	57.4	0.6	18.3
Seepage	21.8	14.9	8.3
Discharges	8.2	3.0	5.4
Total	107.7	27.3	56.7
Deposition	8.9	7.2	8.5
Reduction	9 percent	34 percent	17 percent
Reduction-Conservative Case	–	–	13 percent

DMSTA2 has the capability to provide “conservative” simulations compared to the “base” simulations previously described. The conservative simulation uses a phosphorus renewal rate (k) that is the lowest 10 percentile of the calibrated k values. The conservative simulation indicated a 13 percent reduction in phosphorus for the simulation period compared to a 17 percent reduction for the base simulation.

6.4.2.2 Algae and Dissolved Oxygen

Given the highly variable water depths in the EAA Reservoir A-1 over an annual cycle, phytoplankton (single cell algae) is expected to be the predominate form of algae. The algae growth in the EAA Reservoir A-1 may be desirable because algae help remove phosphorus. Any potential growth of phytoplankton is not expected to interfere with the operation of EAA Reservoir A-1 gate structures or pumps. There is the potential for growth of blue-green algae in the reservoir. Blue-green algae can have excessive growth when conditions are right, such as when high nutrient concentrations exist and when waters are warm and calm. Florida, as well as other states (e.g., Michigan, Oregon, and Virginia) experienced blue-green algae blooms in the summer of 2005. If conditions are right in the waters of the EAA Reservoir A-1, blue-green algae blooms could occur. (See http://www.sfwmd.gov/site/index.php?id=611 for further information.)

Wind energy should be sufficient to keep the normally shallow water vertically well-mixed resulting in dissolved oxygen near saturation concentration most of the time.

6.4.3 Other Water Quality Constituents

The Project Implementation Report (PIR, 2005) identified the other water quality parameters that could be constituents of concern and the potential removal percentage for them in the reservoir. These are summarized in Table 6.4-2.

Table 6.4-2. Predicted Effluent Concentrations (C) For 14 Constituents Other Than P (USACE, 2005)

Parameter	C_{in}	C_{out}	Removal
Turbidity (NTU)	9.43	5.06	46.3%
Total Suspended Solids (mg/L)	13.18	13.18	0.0%
Alkalinity (mg/L as CaCO3)	218.5	73.04	66.6%
Iron (mg/L)	195.9	47.18	75.9%
Calcium (mg/L)	82.67	43.33	47.6%
Sulfate (mg/L)	73.74	58.7	20.4%
Chloride (mg/L)	108.4	107.2	1.1%
Sodium (mg/L)	73.36	46.14	37.1%
Total Nitrogen (mg/L)	3.71	1.79	51.8%
Atrazine (ug/L)	0.429	0.127	70.4%
Total Mercury (ng/L)	2.52	0.179	92.9%
pH (SU)	7.49	7.49	0.0%
Specific Conductivity (uS/cm)	891	891	0.0%
Dissolved Oxygen (mg/L)	4.66	4.66	0.0%

The PIR indicated that there would be no change in the concentration of total suspended solids, pH, specific conductivity, and dissolved oxygen while there would be a decrease in the concentration of the 10 other constituents.

Although not evaluated in detail by this EAA Reservoir A-1 Project, storage in the EAA Reservoir A-1 is not expected to cause or contribute to water quality degradation of these constituents in the NNRC and Miami Canal.

6.4.3 Conclusions

The EAA Reservoir A-1 will not negatively impact water quality in the EAA. Phosphorus contained in the Supply Canals could be removed in the EAA Reservoir A-1 as simulated by the DMSTA2 model. Phytoplankton (single cell algae) will be responsible for most of the phosphorus removal, although chemical precipitation and settling of some phosphorus is possible. However, periodic episodes of floating blue green algae may be aesthetically unpleasing. Significant depletion of dissolved oxygen in the EAA Reservoir A-1 is not expected. Wind energy should be sufficient to keep the water vertically well-mixed resulting in dissolved oxygen near saturation concentration most of the time. During periods when the water surface elevation in the EAA Reservoir A-1 is low, it is possible that wind energy may re-suspend phosphorus from the bottom sediments. DMSTA2 was calibrated using existing water quality data from 11 Florida reservoirs, including Lake Okeechobee, where wind-driven re-suspension of phosphorus occurs. Therefore, DMSTA2 has the capability to calculate the re-suspension of phosphorus in the EAA Reservoir A-1 to evaluate the amount of phosphorous that might be released from EAA Reservoir A-1.

6.5 PUMP STATIONS

6.5.1 Existing Facilities

The EAA Reservoir A-1 will need to work in conjunction with STA-3/4, and existing facilities currently providing service to the STA-3/4 will be incorporated into the operation scheme for the EAA Reservoir A-1. Two existing pump stations currently pump water into the STA-3/4 Supply Canal. Pump station G-370 is equipped with three 925 cfs pumps (total capacity at rated head: 2,775 cfs), and supplies water from the NNRC. Pump station G-372 is equipped with four 925 cfs pumps (total capacity at rated head: 3,700 cfs), and supplies water from the Miami Canal. The rated head is based on the Supply Canal water surface elevation of 13.6 NAVD88. The pumps can operate against a water surface of up to 16.6 NAVD88 at a reduced operating capacity of about 785 cfs (approximately 2,350 cfs total capacity for G-370 pump station, and 3,130 cfs total capacity for G-372 pump station). The water levels in the EAA Reservoir A-1 will fluctuate between elevation 8.6 and 20.6 NAVD88. While it is possible to partially supply the EAA Reservoir A-1 from G-370 and G-372 pump stations without modifications to the pump stations, significant modifications would be required to pump to the full EAA Reservoir A-1 depth. Therefore, all alternatives considered incorporate the existing facilities in one of two operational modes:

- One option is to continue to operate the G-370 and G-372 pump stations without modification and use them to pump into the EAA Reservoir A-1 when storage volume is available at EAA Reservoir A-1 levels lower than elevation of 16.6 NAVD88.

- The other option is to modify the pump stations to pump against a water surface elevation of 20.6 NAVD88, so that the pump stations can be used to pump directly into the EAA Reservoir A-1 at any time that storage volume is available. Four alternatives were evaluated, each of which would allow pumping to the full EAA Reservoir A-1 depth. Each alternative results in a different pumping capacity and is described in Section 13.2 of this report. The resulting alternatives have total pumping capacities of 1,020, 1,860, 2,220, and 2,775 cfs, respectively, for G-370 pump station. Although the same modifications could be applied to G-372 pump station, resulting in capacities of 1,360, 2,480, 2,960, and 3,700 cfs respectively, only the option resulting in 3,700 cfs capacity was considered further to ensure that flood protection capability in the Miami Canal was not compromised.

For either option, the G-370 and G-372 pump stations can be used to pump directly to the STA-3/4 Supply Canal when deliveries to STA-3/4 coincide with flow availability from either the NNRC or the Miami Canal. The primary advantage for the first option is the potential for lower costs; the primary disadvantage is the limited use when the EAA Reservoir A-1 water depth exceeds eight feet. The second option does not have this limitation, but there is significant cost associated with modification to allow pumping to a 12 foot EAA Reservoir A-1 water depth.

6.5.2 Evaluation Process

Pump station sizing was based on a three step process:

- Preliminary Screening – Earlier studies favored the installation of a new pump station located in the northeast corner of the EAA Reservoir A-1 and pumping from the NNRC in combination with existing G-370 and G-372 pump stations. The initial step included a review of various arrangements to determine whether this was still the best option.
- Optimization Based on Deliveries – Based on the results of the preliminary screening process, the WBM was used to establish the minimum pump station capacity required to optimize both environmental and agricultural deliveries from the EAA Reservoir A-1.
- Sizing for Priority Removals – The Water Balance Model (WBM) was used to establish the pump station capacity required to maximize priority withdrawals from the NNRC and the Miami Canal.

The multiple capacity options for G-370 pump station, combined with the fact that optimization for deliveries does not provide the same answer as maximizing priority withdrawals, resulted in a number of options and provides the SFWMD with several viable alternatives with which to proceed. For all alternatives discussed herein, capacities shown are exclusive of seepage pumping needs.

6.5.2.1 Preliminary Screening

Five pumping and discharge alternatives were selected for consideration during a workshop conducted on May 24, 2005. Additional alternatives plus variations to the original alternatives were added later. The original alternatives are designated as Preliminary Screening Alternatives 1, 2, 3, 4 and 5; Preliminary Screening Alternatives 6 and 7 were subsequently added. In general, all alternatives except Alternative 6 are based on the addition of a new northeast pump station located adjacent to the NNRC in the northeast corner of the EAA Reservoir A-1 site. Various alternatives for modifications to existing G-370 and G-372 pump stations are also included. The modification options are described in Section 13.2 of this report. A detailed evaluation of conceptual alternatives is included in Appendix 6-5 (Pumping and Discharge Facilities Technical Memorandum). Table 6.5-1 describes the individual pump station capacities and the total system capacities for the various alternatives described below.

For the preliminary screening process, options were developed only for the needs of the EAA Reservoir A-1. Consideration for additional pumping needs for the EAA Reservoir A-2 was addressed in later evaluation steps. In general, for the preliminary screening process, the facility capacities were developed to complement one or more of the following:

- Existing pumping capacity for both G-370 and G-372 pump stations: In an unmodified condition either pump station could pump directly to the STA-3/4 Supply Canal or to the EAA Reservoir A-1 when EAA Reservoir A-1 water depth was eight feet or less.
- Modified pumping capacity for both G-370 and G-372 pump stations: In a modified condition, both pump stations could pump directly to the STA-3/4 Supply Canal or to the EAA Reservoir A-1 to its full operating depth. This would require additional sitework and infrastructure in addition to the pump station modifications.
- Existing NNRC capacity: Although canal flow capacity was evaluated for two conditions (capacity without velocity restrictions and capacity based on a 2.5 cfs for velocity limit), for clarity, only the first condition is summarized herein. Refer to Appendix 6-2 for further discussion of the second condition.
- Runoff due to local rainfall events: An average allowance of 3/4-inch per day per acre of drainage area was used to estimate agricultural runoff to the NNRC.

The location of the proposed northeast pump station relative to the Bowles and Cross Canals/NNRC interSection and to Lake Okeechobee results in a shorter distance of conveyance than that for G-370 pump station. Consequently, higher canal capacity can be achieved when a greater amount of flow is removed by the northeast pump station. This relationship is illustrated in Figure 6.5-1. In addition, local runoff from precipitation events will also result in shorter conveyance distances than would be experienced in the conveyance of dry weather discharges from Lake Okeechobee and subsequently higher conveyance capabilities will be realized. The difference in dry weather and wet weather capacity is also illustrated in Figure 6.5-1, and can be as much as 35 to 45 percent.

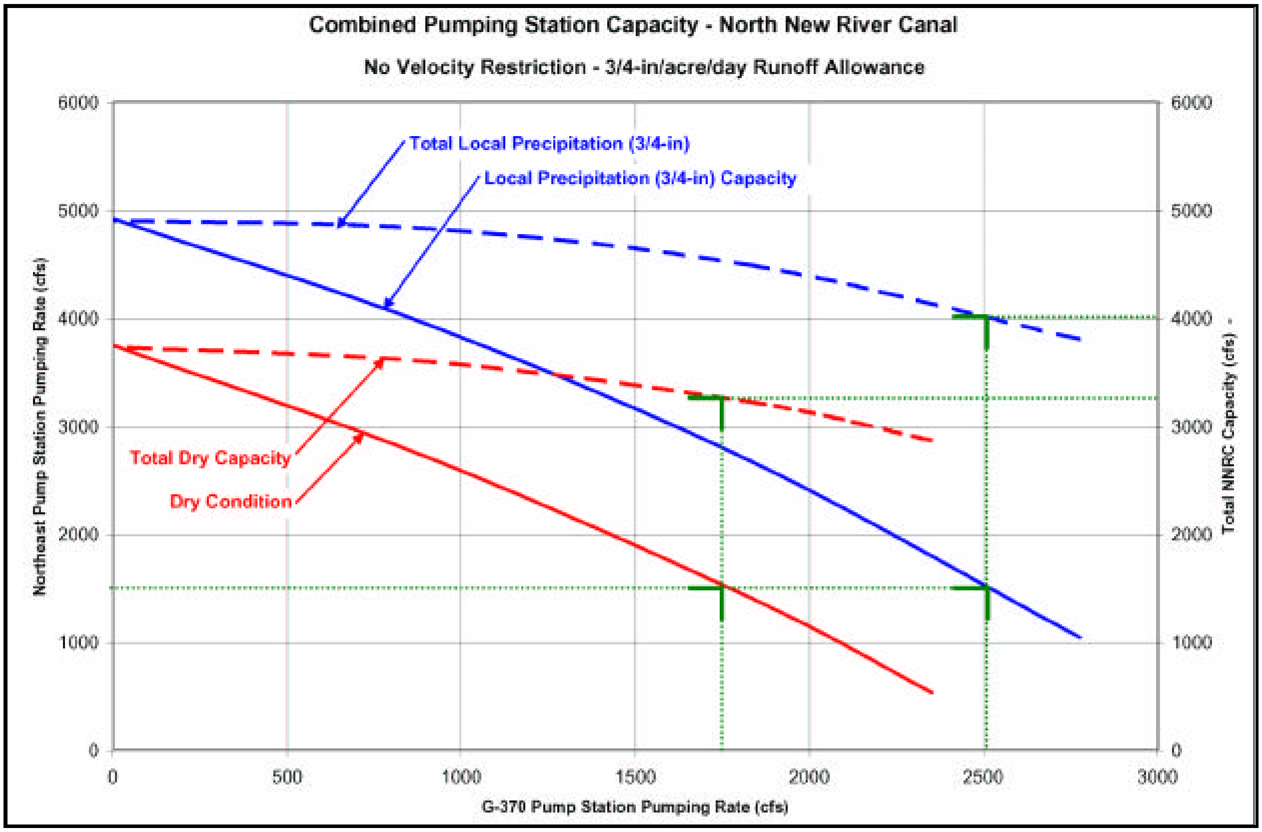
SFWMD has expressed interest in reviewing options that direct all flow through the reservoir prior to discharge into STA-3/4 in order to take advantage of the water quality benefit that may result. To provide this capability, additional structures would be required to completely segregate discharges from both G-370 and G-372 pump stations, directing them without exception to the EAA Reservoir A-1. To demonstrate the cost associated with these additional facilities, Alternatives 2A through 5A were added to the evaluation.

Table 6.5-1 Pump Station Capacities and Total System Capacities

Alternative	G-370 Pump Station Option	Northeast Pump Station Capacity *(cfs)*	G-370 Pump Station Capacity *(cfs)*			G-372 Pump Station Capacity *(cfs)*			Total System Capacity *(cfs)*	Capacity to EAA Reservoir A-1 *(cfs)*
			To STA-3/4 Supply Canal	To Elevation 16.6 NAVD88	To Elevation 20.6 NAVD88	To STA-3/4 Supply Canal	To Elevation 16.6 NAVD88	To Elevation 20.6 NAVD88		To Elevation 20.6 NAVD88
1	A	3,200	2,775	-	-	3,700	-	-	9,675	3,200
2	A	3,200	2,775	2,340	-	3,700	3,120	-	9,675	3,200
3	B	3,800	2,775	-	1,020	3,700	3,120	-	8,520	4,820
	C	2,600	2,775	-	1,860	3,700	3,120	-	8,160	4,460
	D	2,000	2,775	-	2,220	3,700	3,120	-	7,920	4,220
	E	1,000	2,775	-	2,775	3,700	3,120	-	7,475	3,775
4	B	3,000	2,775	-	1,020	3,700	-	3,700	7,720	7,720
	C	2,600	2,775	-	1,860	3,700	-	3,700	8,160	8,160
	D	2,000	2,775	-	2,220	3,700	-	3,700	7,920	7,920
	E	2,000	2,775	-	2,775	3,700	-	3,700	8,475	8,475
5	B	3,800	2,775	-	1,020	3,700	-	3,700	8,520	8,520
	C	2,600	2,775	-	1,860	3,700	-	3,700	8,160	8,160
	D	2,000	2,775	-	2,200	3,700	-	3,700	7,900	7,900
	E	1,000	2,775	-	2,775	3,700	-	3,700	7,475	7,475
6	A	0	2,775	-	2,775*	3,700	-	3,700*	6,475	6,475
7	A	1,000	2,775	-	2,775*	3,700	-	3,700*	7,475	7,475

<u>Notes:</u>
* Transferred to EAA Reservoir A-1 from Supply Canal with a booster pump station with 6,475 cfs capacity.

Figure 6.5-1 Combined Pump Station Capacity - NNRC



NNRC Capacity Example – Northeast Pump Station capacity set at 1,500 cfs.

Dry Conditions – When the northeast pump station is removing 1,500 cfs from the NNRC, the flow available at G-370 pump station is approximately 1,750 cfs (just below the capacity of two of the existing pumps pumping to eight feet of water depth in the EAA Reservoir A-1). Therefore, the total flow available to the EAA Reservoir A-1 from the northeast pump station and the existing G-370 pump station is 3,250 cfs.

Local Precipitation of 3/4-inch Conditions – When the northeast pump station is removing 1,500 cfs from the NNRC, the flow available at G-370 pump station is approximately 2,500 cfs (this is more capacity than the existing G-370 pump station can currently pump to eight feet of water depth). Therefore, the total flow available to the EAA Reservoir A-1 from the northeast pump station and the existing G-370 pump station is 4,000 cfs.

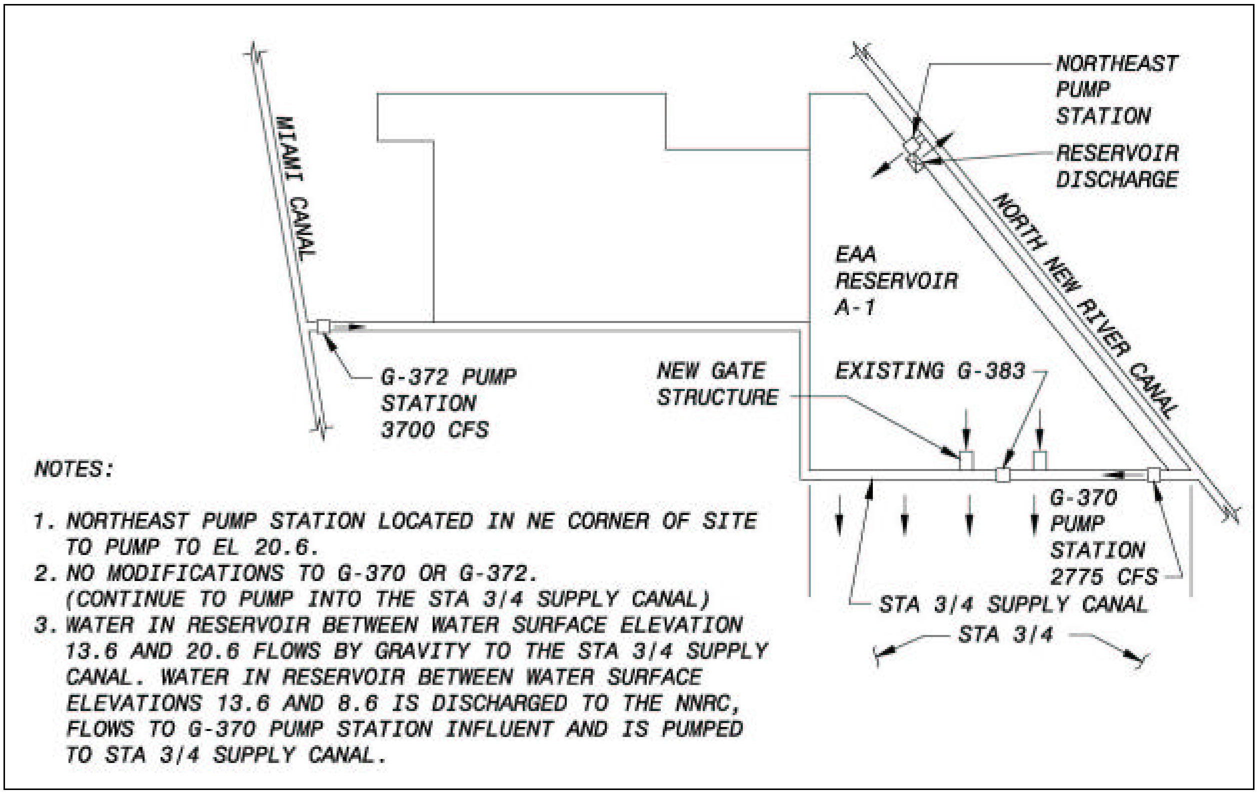
6.5.2.2 Pumping and Discharge Alternatives

6.5.2.2.1 Alternative 1

This alternative, depicted on Figure 6.5-2, includes the installation of a new northeast pump station to exclusively supply water to EAA Reservoir A-1. Under this alternative, the EAA Reservoir A-1 would receive no water from existing G-370 and G-372 pump stations but the existing pump stations would continue to provide service directly to STA-3/4. The new northeast pump station would be designed for an ultimate capacity of 4,900 cfs but would be equipped initially for 3,200 cfs. Space would be provided for the installation of additional pumps in the future.

A gate structure would be located adjacent to the new pump station to allow discharge of flow to the NNRC to meet agricultural deliveries. This northeast gate structure would also be utilized to supply environmental deliveries when the EAA Reservoir A-1 water level is too low for direct discharge into the STA-3/4 Supply Canal. This would be accomplished by discharging to the NNRC and using the G-370 pump station to pump into the STA-3/4 Supply Canal. Two gate structures would be required to supply water to the STA-3/4 Supply Canal from the EAA Reservoir A-1 when water levels are sufficient to allow gravity flow to the supply canal.

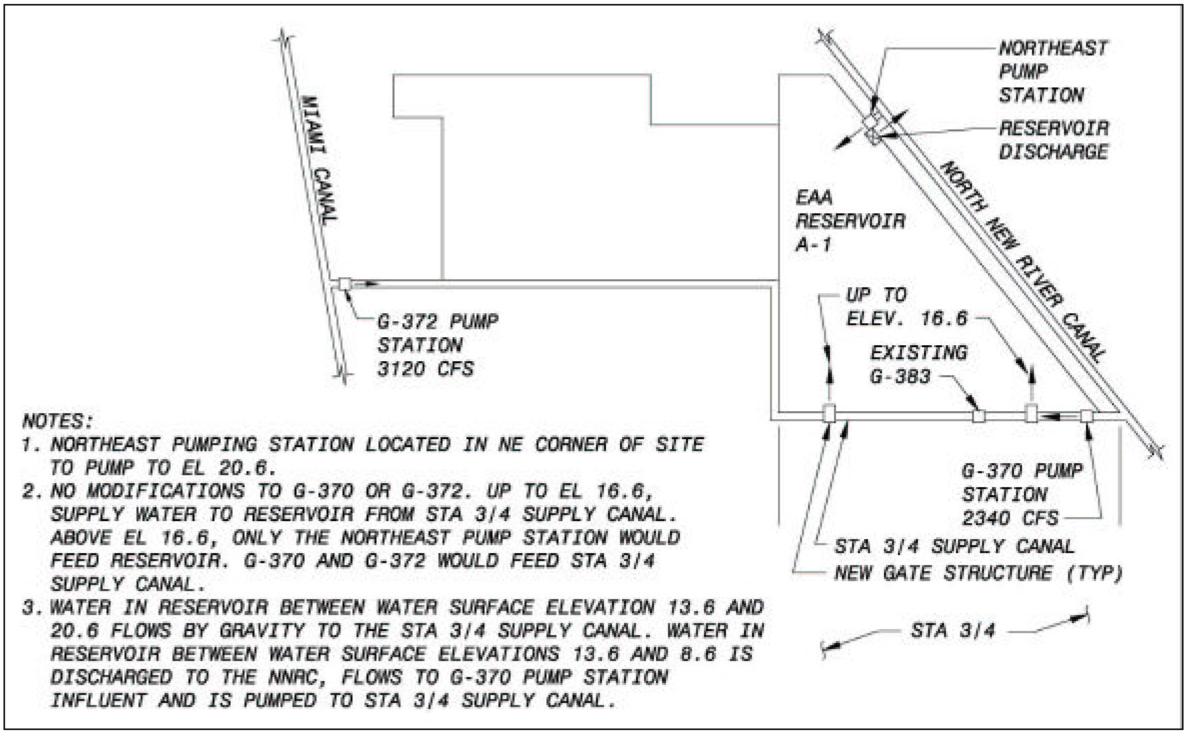
Figure 6.5-2 Preliminary Screening Alternative 1



6.5.2.2.2 Alternatives 2 and 2A

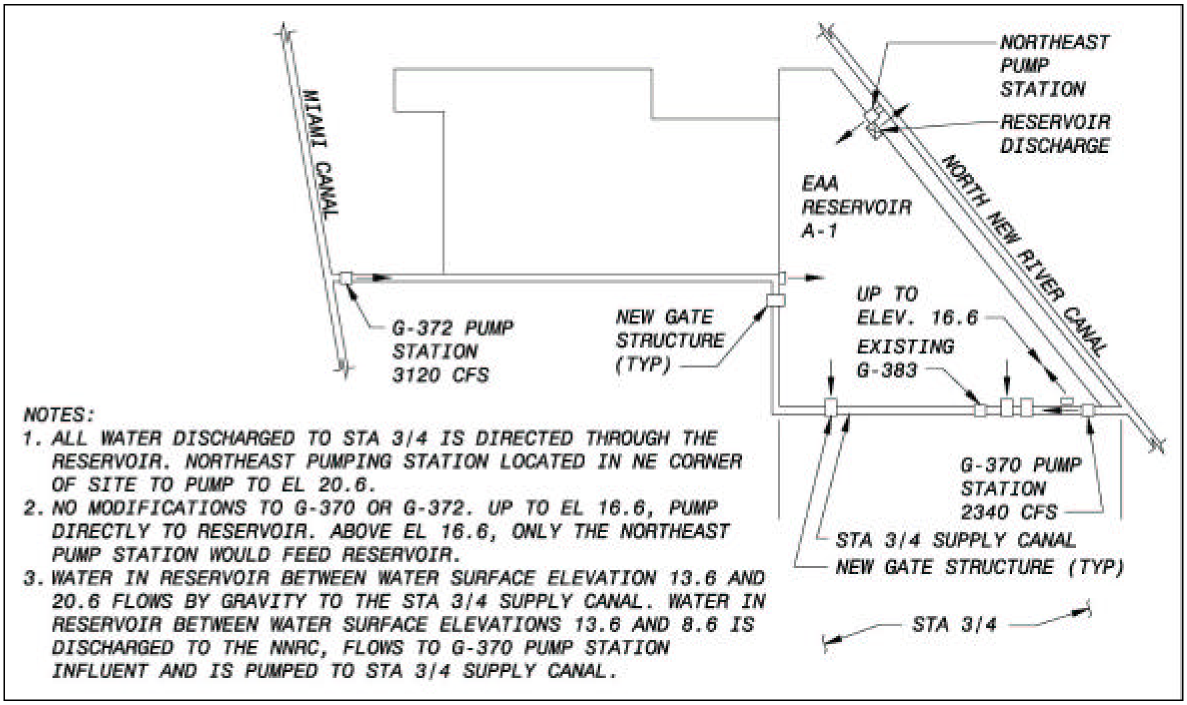
Alternative 2 (Figure 6.5-3) is similar to Alternative 1 with the exception that the G-370 and G-372 pump stations would be used without modifications to supply EAA Reservoir A-1 when its water depths are lower than an elevation of 16.6 NAVD88. A new northeast pump station would discharge to the EAA Reservoir A-1 over the full 12-foot operating range. Gate structures would be located between the STA-3/4 Supply Canal and EAA Reservoir A-1 to serve a dual purpose of EAA Reservoir A-1 filling and discharge. Once the water level in the EAA Reservoir A-1 exceeds an elevation of 16.6 NAVD88, all flow into the EAA Reservoir A-1 would be from the northeast pump station, and G-370 and G-372 pump stations would continue to provide water directly to STA-3/4 through the Supply Canal.

Figure 6.5-3 Preliminary Screening Alternative 2



Alternative 2A (Figure 6.5-4) involves adding four gate structures in addition to those required for Alternative 2 to limit discharge from the G-370 and G-372 pump stations only to the EAA Reservoir A-1 so that there is no direct discharge from either of these pump stations to the STA-3/4 Supply Canal.

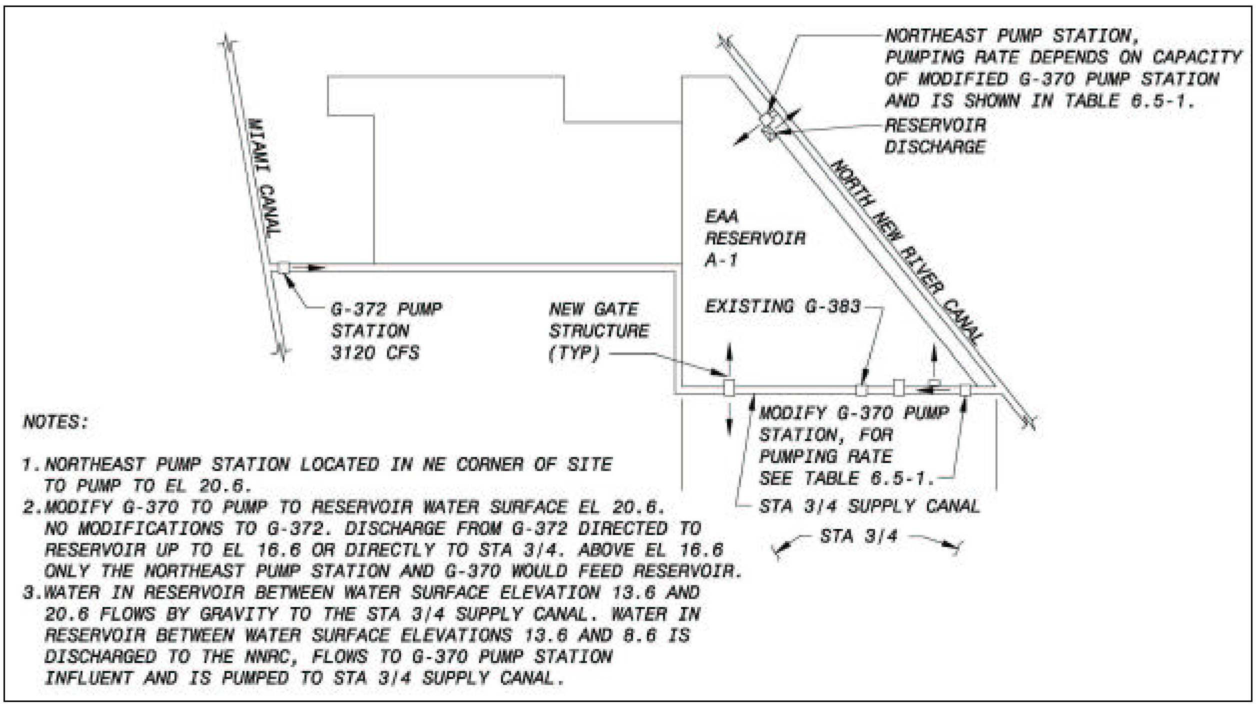
Figure 6.5-4 Preliminary Screening Alternative 2A



6.5.2.2.3 Alternatives 3 and 3A

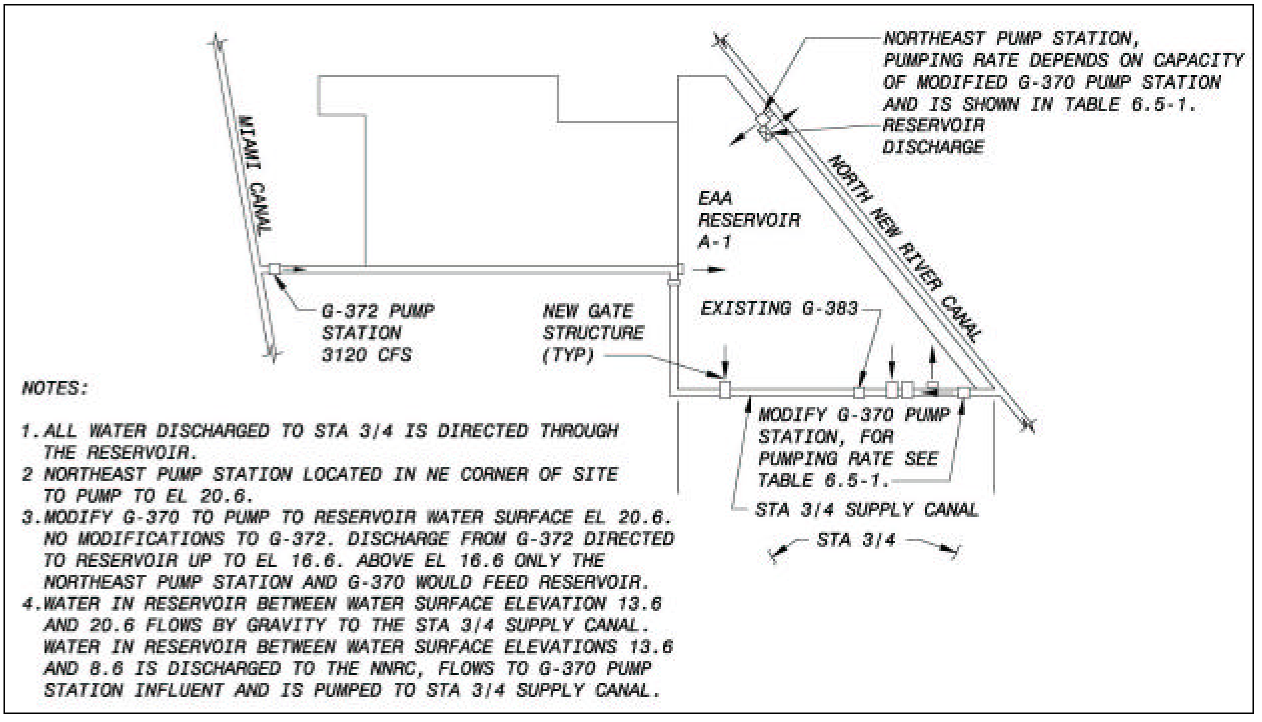
Alternative 3 (Figure 6.5-5) includes the installation of a new northeast pump station combined with modifications to G-370 pump station sufficient to allow pumping from this station to a full EAA Reservoir A-1 elevation of 20.6 NAVD88. There are a number of options available, with each modification becoming progressively more complicated and expensive but resulting in greater capacity. The resulting capacities for the G-370 pump station range from 1,020 cfs to 2,775 cfs and are summarized in Table 6.5-1. Because the Supply Canal and EAA Reservoir A-1 have different maximum operating levels, modifications to the Supply Canal would be required to allow diversion of the G-370 pump station flow into the EAA Reservoir A-1. To match the existing capacity of the NNRC, the capacity of the northeast pump station decreases as the capacity of the G-370 pump station increases. In this alternative, there are no modifications to the G-372 pump station. Therefore, it would be used to pump to the Supply Canal with discharge either directly to STA-3/4 or to the EAA Reservoir A-1 when water levels are below elevation of 16.6 NAVD88.

Figure 6.5-5 Preliminary Screening Alternative 3



Alternative 3A (Figure 6.5-6) involves adding the same four gate structures described for Alternative 2A to pump all water from the G-370 and G-372 pump stations into the EAA Reservoir A-1 while all water delivered to the environment is discharged from the EAA Reservoir A-1. Because this alternative includes no modification to G-372 pump station, and, under this scenario, G-372 pump station would not be allowed to pump directly to STA-3/4, the pump station could only be operated when EAA Reservoir A-1 water surface elevations are less than 16.6 NAVD88. Flows from the Miami Canal could not be pumped when EAA Reservoir A-1 water surface elevations exceeded 16.6 NAVD88. Therefore, this alternative is not a viable option.

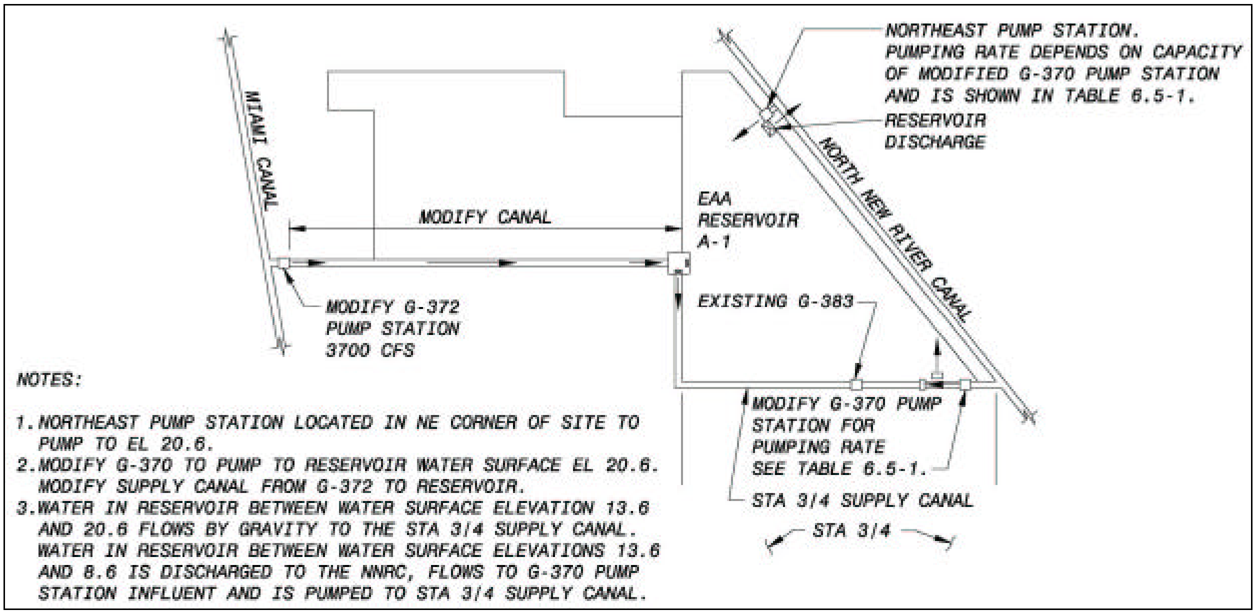
Figure 6.5-6 Preliminary Screening Alternative 3A



6.5.2.2.4 Alternatives 4 and 4A

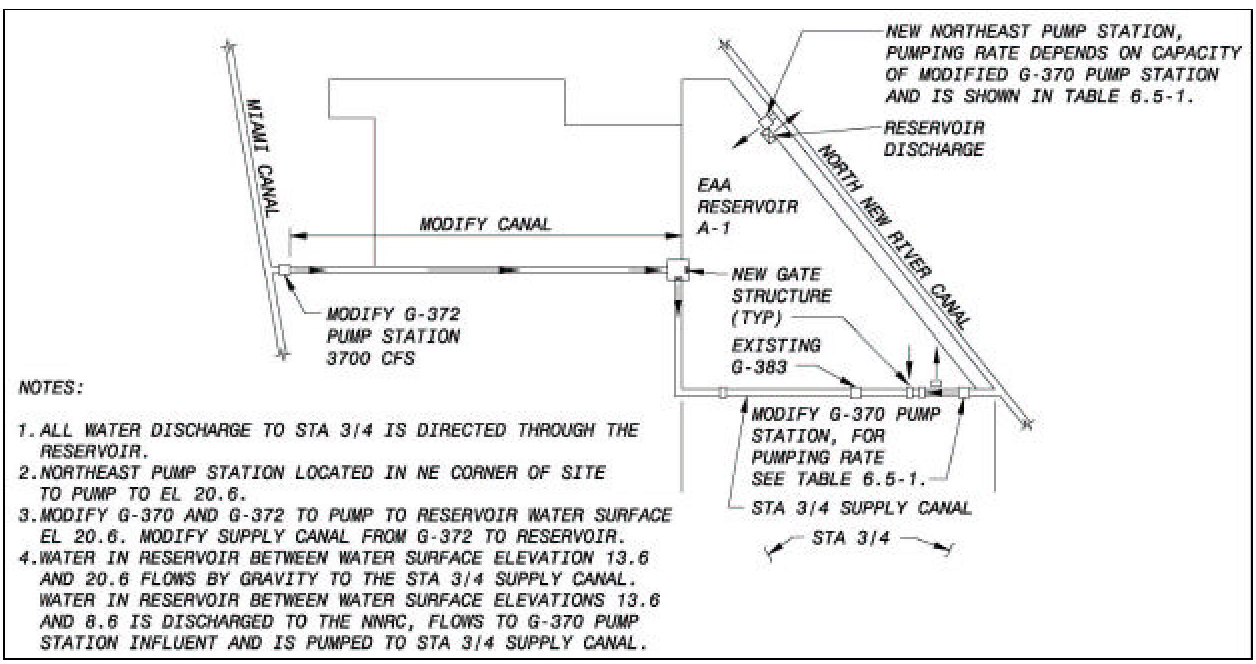
Alternative 4 (Figure 6.5-7) is similar to Alternative 3 except that it expands that alternative by modifying structure G-372 pump station to pump to the full EAA Reservoir A-1 depth (elevation of 20.6 NAVD88) and by increasing the levee height for the Supply Canal from G-372 pump station to EAA Reservoir A-1. Because G-372 pump station alone serves the Miami Canal, it was assumed that modifications to the pump station would result in the full 3,700 cfs capacity currently experienced so that flood protection capability would not be diminished. Under this alternative, the Supply Canal control structures would serve a dual function of both EAA Reservoir A-1 inlet and outlet.

Figure 6.5-7 Preliminary Screening Alternative 4



Alternative 4A (Figure 6.5-8) involves adding two gate structures dedicated to withdrawing water from EAA Reservoir A-1 to the Supply Canal. Unlike Alternative 3A, G-372 pump station can be used at all times that there is available EAA Reservoir A-1 capacity; therefore, this would be a viable option.

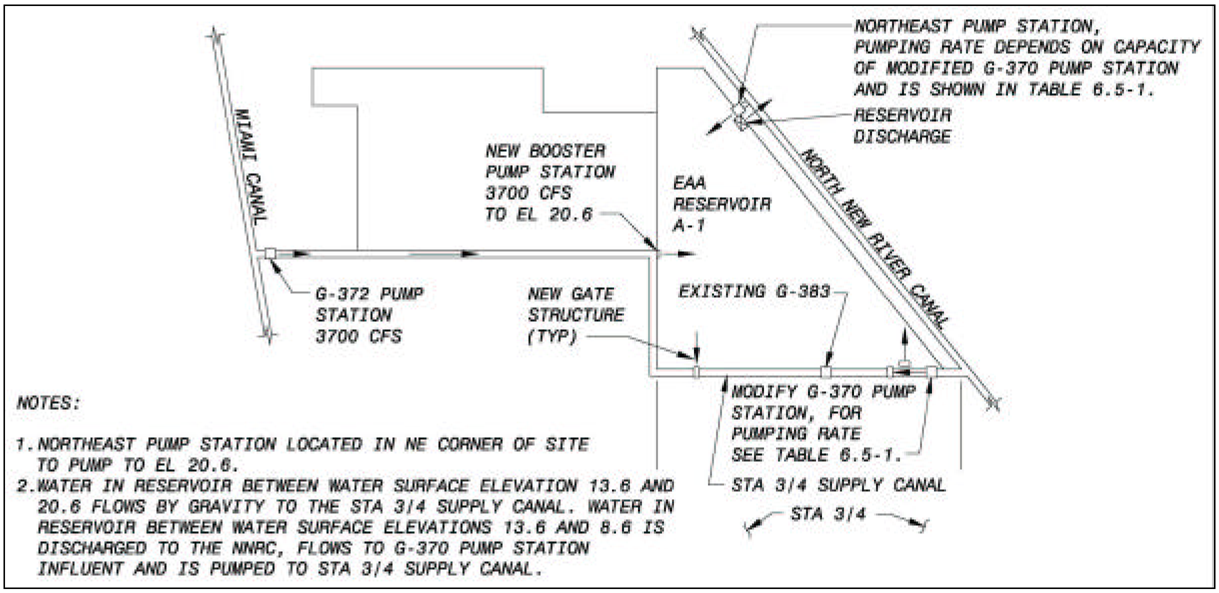
Figure 6.5-8 Preliminary Screening Alternative 4A



6.5.2.2.5 Alternatives 5 and 5A

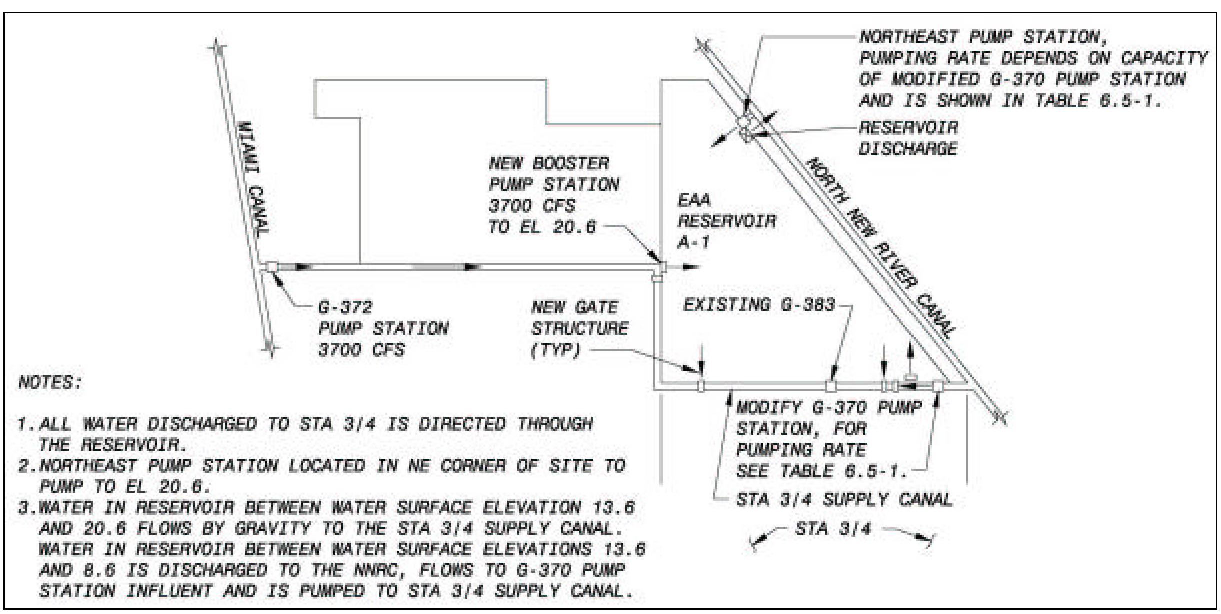
Alternative 5 (Figure 6.5-9) is similar to Alternative 4 in all aspects except that rather than modifying G-372 pump station and the Supply Canal between the pump station and the EAA Reservoir A-1, a new booster pump station would be located in the southeast corner of the EAA Reservoir A-1 to boost the 3,700 cfs flow from G-372 pump station to the EAA Reservoir A-1 water surface elevation of 20.6 NAVD88. All other features of Alternative 4 would be included in this alternative.

Figure 6.5-9 Preliminary Screening Alternative 5



Alternative 5A (Figure 6.5-10) involves adding a gate structure near G-370 pump station to release water from EAA Reservoir A-1 to the STA-3/4 Supply Canal and a gate structure on the Supply Canal downstream of the new booster pumping station.

Figure 6.5-10 Preliminary Screening Alternative 5A

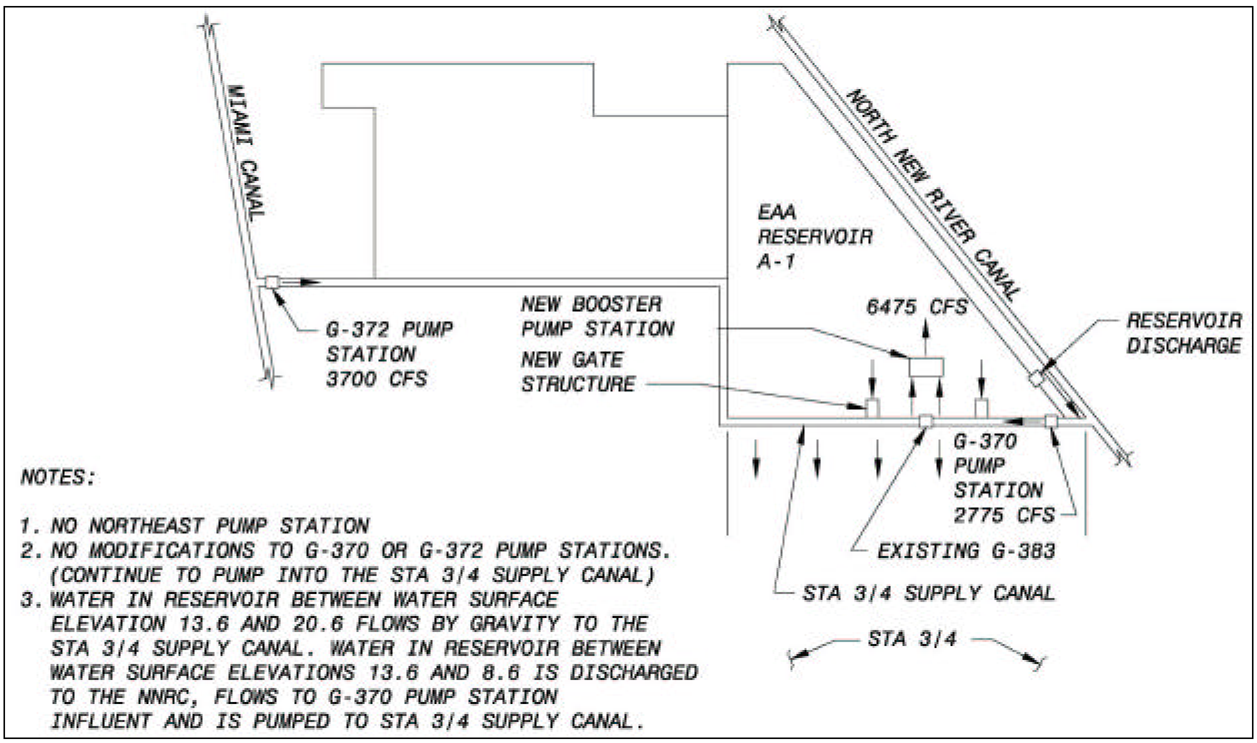


6.5.2.2.6 Alternative 6

Alternative 6 (Figure 6.5-11) retains G-370 and G-372 pump stations in their current state and includes a new booster station to pump water from the STA-3/4 Supply Canal into EAA Reservoir A-1 up to its full elevation of 20.6 NAVD88. No additional pumping capacity would from the NNRC would be provided. Gate structures would be provided to discharge water from the EAA Reservoir A-1 to the Supply Canal when the EAA Reservoir A-1 water surface elevation is above an elevation of 13.6 NAVD88. A second gate structure would be required to release water from the EAA Reservoir A-1 to the NNRC for agricultural deliveries and for environmental deliveries when the EAA Reservoir A-1 level is insufficient to discharge directly to the Supply Canal.

The relative location of the booster pump station to the existing G-370 and G-372 pump stations, the supply canal, and STA-3/4 influent structures precludes directing all flow to the reservoir prior to discharge to STA-3/4. Therefore, no Alternative 6A was considered. (This could be accomplished with two separate booster pump stations size at 2,775 cfs and 3,700 cfs respectively serving G-370 and G-372 pump stations. The cost for the two stations would be greater than that for a single pump station, so this option was excluded from further consideration.)

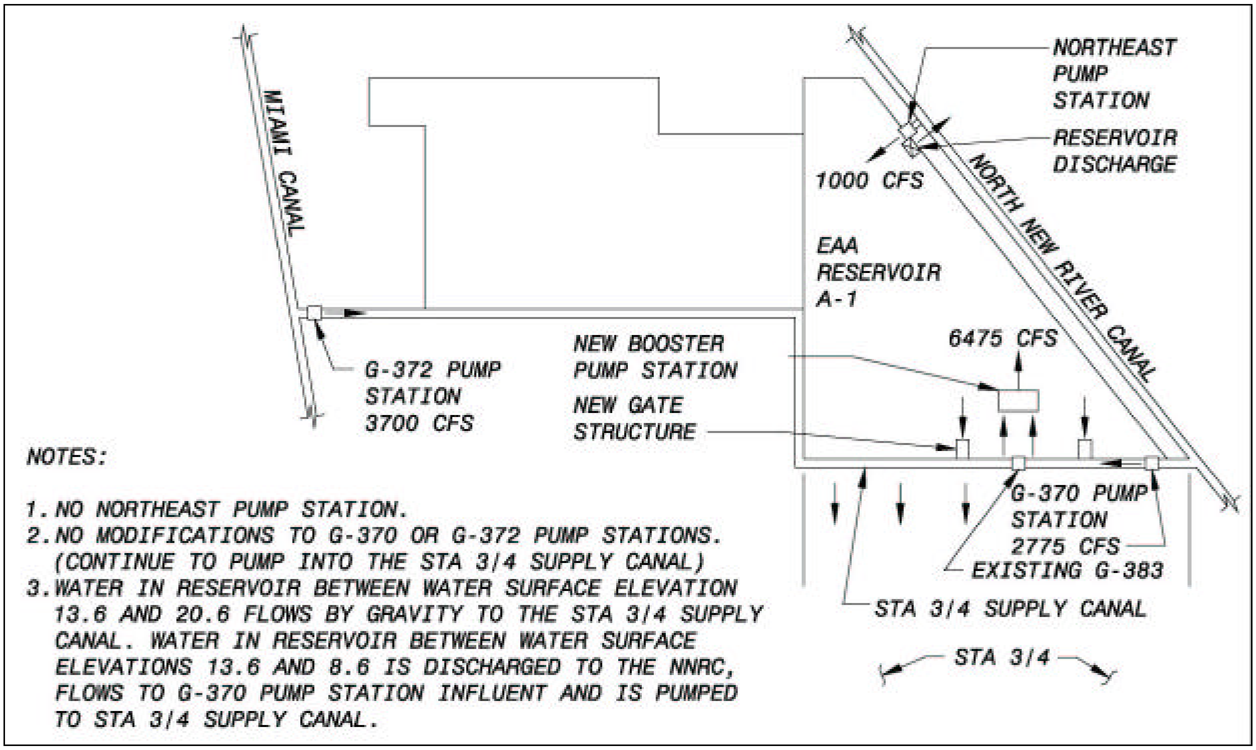
Figure 6.5-11 Preliminary Screening Alternative 6



6.5.2.2.7 Alternative 7

Alternative 7 (Figure 6.5-12) is similar to Alternative 6, except that a 1,000 cfs northeast pump station is included in order to take full advantage of the existing capacity of the NNRC during local precipitation events.

Figure 6.5-12 Preliminary Screening Alternative 7



6.5.2.2.8 Discussion of Alternatives

Advantages and disadvantages for each alternative are listed in Table 6.5-2. The costs associated with each alternative are shown in Tables 6.5-3 and 6.5-4.

In general, the cost for the additional infrastructure required to direct all flows through the EAA Reservoir A-1 prior to discharge to the STA-3/4 Supply Canal ranges from \$15M to \$35M. Although there may be some treatment value associated with EAA Reservoir A-1 storage, flow management is the EAA Reservoir A-1's primary function; treatment is a secondary benefit. The STA-3/4 is designed to sufficiently treat flows pumped directly from the canals without the benefit of treatment that might be experienced with the EAA Reservoir A-1. Due to the additional cost associated with the 2A, 3A, 4A, and 5A alternatives and the limited potential benefit, we recommend against further consideration of these alternatives.

The most cost effective alternatives are 1, 2, 3E, and 6. While Alternative 6 is the lowest cost option, the primary disadvantage is that it offers no increased pumping capacity from the NNRC and therefore no increased flood protection. Although it is feasible that this pump station could be combined with a future northeast pump station augmented by increased canal capacity at the time that the EAA Reservoir A-2 is constructed, the location would not favor the proposed two cell operation (with this arrangement all pumping would be in to the EAA Reservoir A-1).

The cost difference between Alternatives 1 and 2 are nominal. The limited use of the G-370 and G-372 pump stations in Alternative 1 would favor the selection of Alternative 2 over Alternative 1 considering the relatively small cost difference.

For the first phase of the EAA Reservoir A-1 construction and operation (cell A1), Alternatives 2 and 3 are preferred over the others. While Alternative 3 is cost competitive with Alternative 2, modifications to the existing pump stations that would be required for Alternative 3 would be difficult and potentially disruptive to the continuous operation of STA-3/4. The primary limitation for Alternative 2 is that the existing G-370 and G-372 pump stations would only be useful for filling the EAA Reservoir A-1 to approximately eight feet of depth or for pumping directly to the STA-3/4 Supply Canal. At any time that the EAA Reservoir A-1 has a water depth greater than eight feet, all flow into the EAA Reservoir A-1 must be supplied from the new northeast pump station. Implementation of either alternative would significantly increase the flood pumping capability for the system.

All further refinements in pump station capacity focused on Alternatives 2 and 3.

Table 6.5-2 Advantages and Disadvantages for each Pump Station Alternative

Alternative	Advantages	Disadvantages
Alternative 1	• Increase pumping capacity from NNRC. • No change to existing pump stations (therefore, no associated costs nor operational impact during construction).	• G-370 and G-372 pump stations have no capability to pump flow into EAA Reservoir A-1, therefore limited to pump to STA-3/4 • Water pumped from G-370 and G-372 pump stations does not pass through EAA Reservoir A-1 (no treatment value).
Alternative 2	• Increase pumping capacity from NNRC • No change to existing pump stations (therefore, no associated costs nor operational impact during construction).	• Above elevation 16.6 NAVD88, decreased pumping capacity to EAA Reservoir A-1, water pumped from G-370 and G-372 pump stations does not pass through EAA Reservoir A-1 (no treatment value). • Increased operator attention required (due to switch over at elevation 16.6 NAVD88).
Alternative 3	• Increase pumping capacity from NNRC • No change to existing pump stations (therefore, no associated costs nor operational impact during construction). • All available flow from NNRC pumped through EAA Reservoir A-1 (unless EAA Reservoir A-1 has no remaining capacity). • G-372 pump stations utilized for elevation 16.6 NAVD88 and under. Available for pumping under flood conditions.	• Increased operator attention required (due to switch over at elevation 18 NAVD88). • Above elevation 16.6 NAVD88, pumping from G-372 pump station limited to STA-3/4 • Requires modification of G-370 pump station to allow pumping directly to EAA Reservoir A-1 • Heightened degree of difficulty - need to maintain G-370 pump station in operation during construction.

Alternative	Advantages	Disadvantages
Alternative 4	• Increase pumping capacity from NNRC • All available flow from NNRC and Miami canal can be pumped to EAA Reservoir A-1 (unless EAA Reservoir A-1 has no remaining capacity)	• High Cost • Requires modification of G-370 pump station to allow pumping directly to EAA Reservoir A-1 • Requires modification of G-372 pump station and adjacent feeder canal to allow pumping directly to EAA Reservoir A-1 • Heightened degree of difficulty - need to maintain G-370 pump station in operation during construction
Alternative 5	• Increase pumping capacity from NNRC • All available flow from NNRC and Miami pumped through EAA Reservoir A-1 (unless EAA Reservoir A-1 has no remaining capacity) • Simple operation	• High cost • G-372 booster pumping station difficult to access • Heightened degree of difficulty - need to maintain G-370 pump station in operation during construction
Alternative 6	• Utilizes existing pump stations with no further modification • Simple construction • Booster pump station could be designed to pump into and out of EAA Reservoir A-1 (to STA-3/4 Supply Canal) • Lowest cost	• No increase in pumping capacity from NNRC • Water pumped from G-370 and G-372 pump stations does not pass through EAA Reservoir A-1 if directed to STA-3/4 (no treatment value) • Booster pump station difficult to access
Alternative 7	• Utilizes existing pump stations with no further modification • Simple construction • Increase pumping capacity from NNRC • Booster pumping station could be designed to pump into and out of EAA Reservoir A-1 (to STA-3/4 Supply Canal)	• Water pumped from G-370 and G-372 pump stations does not pass through EAA Reservoir A-1 if directed to STA-3/4 (no treatment value) • Requires construction of two pump stations • High cost • Booster pump station difficult to access

Table 6.5-3 Cost Estimations for Each Pump Station Alternatives 1-7

			G-370 Pump Station Modifications		G-372 Pump Station Modifications			
Alternative	**G-370 Pump Station Option**(1)	**Northeast Pump Station** (million)	**Mechanical and Miscellaneous Modifications** (million) (2)	**Civil** (million) (3)	**Mechanical and Miscellaneous Modifications** (million) (2)	**Civil** (million) (3)	**Booster Pump Station** (million)	**Total** (million)
1		$83.0	-	$6.9	-	$7.1	-	$97.0
2		$83.0	-	$6.9	-	$7.1	-	$97.0
3	B	$86.0	$3.6	$33.7	-	$7.1	-	$130.4
	C	$61.5	$10.6	$33.7	-	$7.1	-	$112.9
	D	$56.0	$12.2	$33.7	-	$7.1	-	$109.0
	E	$44.5	$14.9	$33.7	-	$7.1	-	$100.2
4	B	$86.0	$3.6	$33.7	$4.6	$64.2	-	$192.1
	C	$61.5	$10.6	$33.7	$13.1	$64.2	-	$183.1
	D	$56.0	$12.2	$33.7	$15.2	$64.2	-	$181.3
	E	$44.5	$14.9	$33.7	$20.1	$64.2	-	$177.4
5	B	$86.0	$3.6	$33.7	-	$7.1	$42.7	$173.1
	C	$61.5	$10.6	$33.7	-	$7.1	$42.7	$155.6
	D	$56.0	$12.2	$33.7	-	$7.1	$42.7	$151.7
	E	$44.5	$14.9	$33.7	-	$7.1	$42.7	$142.9
6		-	-	$6.9	-	$7.1	$67.6	$81.6
7		$44.5	-	$6.9	-	$7.1	$67.6	$126.1

(1) Refer to Table 6.5-1 Pump Station Capacities and Total System Capacities for definitions of each option
(2)Miscellaneous modifications include mechanical and structural modifications to existing seepage pumps
(3) Civil modifications include discharge modifications at pump station structures

Table 6.5-4 Cost Estimations for Each Pump Station Alternatives 2A – 5A

Alternative	G-370 Pump Station Option*	Northeast Pump Station (million)	G-370 Pump Station Modifications: Mechanical and Miscellaneous Modifications (million) (2)	G-370 Pump Station Modifications: Civil (million) (3)	G-372 Pump Station Modifications: Mechanical and Miscellaneous Modifications (million) (2)	G-372 Pump Station Modifications: Civil (million) (4)	Booster Pump Station (million)	Total (million)
2A		$83.0	-	$18.2	-	$24.1	-	$125.3
3A	B	$86.0	$3.6	$39.3	-	$24.1	-	$153.0
	C	$61.5	$10.6	$39.3	-	$24.1	-	$135.5
	D	$56.0	$12.2	$39.3	-	$24.1	-	$131.6
	E	$44.5	$14.9	$39.3	-	$24.1	-	$122.8
4A	B	$86.0	$3.6	$39.3	$4.6	$71.3	-	$204.8
	C	$61.5	$10.6	$39.3	$13.1	$71.3	-	$195.8
	D	$56.0	$12.2	$39.3	$15.2	$71.3	-	$194.0
	E	$44.5	$14.9	$39.3	$20.1	$71.3	-	$190.1
5A	B	$86.0	$3.6	$39.3	-	$24.1	$42.7	$195.7
	C	$61.5	$10.6	$39.3	-	$24.1	$42.7	$178.2
	D	$56.0	$12.2	$39.3	-	$24.1	$42.7	$174.3
	E	$44.5	$14.9	$39.3	-	$24.1	$42.7	$165.5

(1) Refer to Table 6.5-1 Pump Station Capacities and Total System Capacities for definitions of each option
(2)Miscellaneous modifications include mechanical and structural modifications to existing seepage pumps
(3) Civil modifications include discharge modifications at pump station structures and southeast gate structure
(4) Civil modifications include discharge modifications at pump station structures and southwest gate structure

6.5.2.3 Optimization Based on Water Deliveries

One of the primary goals of the EAA Reservoir A-1 is to store and equalize flow to improve deliveries to the environment and agricultural users. Therefore, the first refinement analyzed was to optimize pump station capacity based on water deliveries. The WBM described in Section 6.2 was used to determine the minimum capacity that would be needed for each of the three pump stations (northeast, G-370, and G-372) to optimize the number and volume of deliveries over the period of simulation. For this analysis, it was assumed that the NNRC would be modified as required to provide sufficient capacity for each alternative analyzed and would not be a limitation. The associated canal improvements necessary for the alternatives are discussed in Section 10.

Because there are three pump stations involved, each of which could have a variable capacity, several rules were established to limit the final number of options evaluated:

- Pumping options for G-370 pump station were based on the options identified in the preliminary screening process and described in Section 13:
  - No modification, capacity of 2,775 cfs when pumping to the Supply Canal; capacity of 2,350 cfs when pumping to a EAA Reservoir A-1 depth of 8 feet
  - Modified, capacity of 2,775 cfs when pumping to the Supply Canal, capacity of 1,020 cfs when pumping to a EAA Reservoir A-1 depth up to 12 feet
  - Modified, capacity of 2,775 cfs when pumping to the Supply Canal, capacity of 1,860 cfs when pumping to a EAA Reservoir A-1 depth up to 12 feet
  - Modified, capacity of 2,775 cfs when pumping to the Supply Canal, capacity of 2,220 cfs when pumping to a EAA Reservoir A-1 depth up to 12 feet
  - Modified, capacity of 2,775 cfs when pumping to the Supply Canal, capacity of 2,775 cfs when pumping to a EAA Reservoir A-1 depth up to 12 feet
- Pumping options for G-372 pump station were limited to those that maintained a pumping rate of 3,700 cfs so that flood protection would not be diminished. Reduced rate options were identified but were removed from further consideration.
  - No modification, capacity of 3,700 cfs when pumping to the Supply Canal; capacity of 3,130 cfs when pumping to a EAA Reservoir A-1 depth up to 8 feet
  - Modified, capacity of 3,700 cfs when pumping to the Supply Canal, capacity of 3,700 cfs when pumping to an EAA Reservoir A-1 depth of 12 feet
- Pumping capacities for the northeast pump station were varied to establish the minimum capacity required to work in combination with the options described above for G-370 and G-372 pump stations.

For the resulting alternatives, the WBM was applied using several different flow/delivery conditions as defined by the SFWMD ECP:

- 2010 water availability, 2010 environmental and agricultural deliveries
- 2010 water availability, 2015 environmental deliveries and 2010 agricultural deliveries
- 2015 water availability, 2015 environmental and agricultural deliveries

The first two conditions were applied to an EAA Reservoir configuration that consisted of the A-1 cell only. The last condition was applied to an EAA Reservoir configuration that consisted of both the A-1 and A-2 cells.

Water balance models were developed and run as follows:

1) EAA-A1.xls – This models the EAA Reservoir A-1 with 2010 environmental and agricultural deliveries. The G-370 and G-372 pump stations are not modified and pump only to 8 feet of EAA Reservoir A-1 depth.
2) EAA-A1_2015_Envtl&Flows.xls – This models the EAA Reservoir A-1 with 2015 environmental and agricultural deliveries. The G-370 and G-372 pump stations are not modified and pump only to eight feet of EAA Reservoir A-1 depth.
3) EAA-A1_2015_Envtl.xls – This models the EAA Reservoir A-1 with 2015 environmental deliveries and 2010 agricultural deliveries met after 2015 environmental calls. The G-370 and G-372 pump stations are not modified and pump only to eight feet of EAA Reservoir A-1 depth.
4) EAA-A1_G-370_Modified.xls – This models the EAA Reservoir A-1 with 2010 environmental and agricultural calls. The G-370 pump station is modified and pumps to 12 feet of EAA Reservoir A-1 depth while the G-372 pump station is not modified and pumps to only eight feet of EAA Reservoir A-1 depth.
5) EAA-A1_G-370_Modified_2015_Envtl.xls – This models the EAA Reservoir A-1 with 2015 environmental deliveries and 2010 agricultural deliveries met after 2015 environmental deliveries. The G-370 pump station is modified and pumps to 12 feet of EAA Reservoir A-1 depth while the G-372 pump station is not modified and pumps to only eight feet of EAA Reservoir A-1 depth.
6) EAA-A1+A2_2015_Flows&Demands.xls – This models both the EAA Reservoir A-1 and EAA Reservoir A-2, and therefore includes 2015 flows, environmental deliveries, and agricultural calls. Pump stations G-370 and G-372 are not modified and pump only to 8 feet of EAA Reservoir A-1 depth.
7) EAA-A1+A2_G-370&G-372_Modified_2015_Flows&Demands.xls – This models both the EAA Reservoir A-1 and EAA Reservoir A-2, and therefore, includes 2015 flows, environmental deliveries, and agricultural deliveries. The G-370 and G-372 pump stations are both modified to pump to 12 feet of EAA Reservoir A-1 depth.

The features of each water balance model are summarized in matrix form in Table 6.5-5 below.

Table 6.5-5 Summary of Water Balance Models' Features

WBM No.	Environmental Demands		Agricultural Demands		G-370 pump station pumping depth		G-372 pump station pumping depth	
	2010	2015	2010	2010*	8-foot	12-foot	8-foot	12-foot
1	X		X		X		X	
2		X		X	X		X	
3		X	X		X		X	
4	X		X			X	X	
5		X	X			X	X	
6		X		X	X		X	
7		X		X		X		X

* 2010 agricultural demands after 2015 environmental demands have already been met

Figures 6.5-13 through 6.5-23 provide summaries for each alternative.

The summaries identify the percent of the deliveries that can be achieved along with associated costs. Table 6.5-6 provides an overall summary to allow direct comparison of the alternatives.

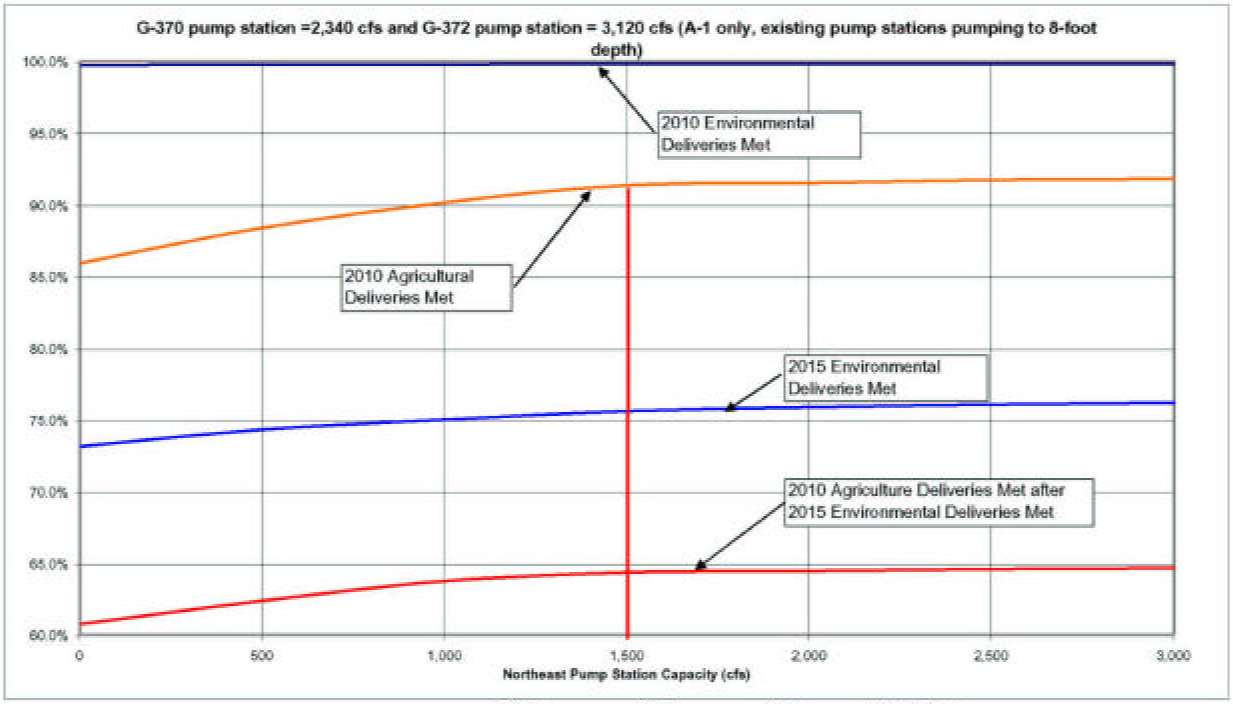
For the first phase of the EAA Reservoir A-1 construction, it was assumed that there would be no modification to G-372 pump station based on the associated high cost identified during the preliminary screening. However, in the event that construction of the EAA Reservoir A-2 was delayed, model runs were also conducted based on 2010 flow availability and 2015 environmental delivery conditions. From Table 6.5-6 it appears that a combined capacity of the northeast pump station and G-370 pump station in the range of 2,500 to 3,000 cfs would be sufficient to allow optimum deliveries based on the EAA Reservoir A-1 capacity available, the flows available (2010), and either the 2010 or 2015 environmental delivery condition. The lowest cost alternative is A1-1A, a 1,500 cfs northeast pump station working in conjunction with an unmodified G-370 pump station. Although G-370 pump station would have a capacity of up to 2,350 cfs when all three pumps are pumping up to an 8 foot EAA Reservoir A-1 depth, it is apparent from comparison of Figures 6.5-13 and 6.5-14 that the same delivery percentages can be gained with two pumps operating at 1,565 cfs capacity. The combined capacity of the two stations is within the current capacity limits of the NNRC. Consequently, this alternative would not require canal modifications at this time to achieve the optimum deliveries.

For the second phase of the EAA Reservoir construction, Alternative A2-2C matches the capacity for the northeast pump station identified above for phase one with one of the modification options for G-370 pump station. Minor canal modifications would be needed to ensure that there would be sufficient flow available to meet the 2015 delivery needs.

Table 6.5-6 Costs Associated with Pump Station Alternatives and Options Based on Deliveries Met

		Capacity														
Option	Description	Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Combined Capacity - Northeast Pump Station and G-370 Pump Station	EAA Reservoir	Flow Condition	Delivery Condition	Canal Improvement	Cost ($Millions)	Optimization Goal	Envtl Delivery % Met	Irrigation Delivery % Met	Runoff Captured % Stored	Back Pumping % Captured	Comment
A1-1A	G-370 and G-372 pump stations unmodified	1,500	1,560	3,120	3,060	A-1	2010	2010 & 2015	No	$74.3	Deliveries	99.7	91.3	-	-	Appears to provide the optimum delivery at lowest cost
A1-2B	G-370 pump station modified to 12', G-372 pump station unmodified	1,500	1,020	3,120	2,520	A-1	2010	2010 & 2015	No	$103.0	Deliveries	99.7	91.2	-	-	Increased cost over Option A1-1A, with no significant increase in deliveries (approximately the same level of delivery achieved)
A1-2C	G-370 pump station modified to 12', G-372 pump station unmodified	1,000	1,860	3,120	2,860	A-1	2010	2010 & 2015	No	$106.4	Deliveries	99.7	91.3	-	-	Increased cost over Option A1-1A, with no significant increase in deliveries (approximately the same level of delivery achieved)
A1-2D	G-370 pump station modified to 12', G-372 pump station unmodified	500	2,220	3,120	2,720	A-1	2010	2010 & 2015	No	$97.4	Deliveries	99.7	91.3	-	-	Increased cost over Option A1-1A, with no significant increase in deliveries (approximately the same level of delivery achieved)
A1-2E	G-370 pump station modified to 12', G-372 pump station unmodified	0	2,775	3,120	2,775	A-1	2010	2010 & 2015	Yes	$91.9	Deliveries	99.7	91.3	-	-	Increased cost over Option A1-1A, with no significant increase in deliveries (approximately the same level of delivery achieved)
A2-2B	G370 and G 372 modified to pump to 12'	2,000	1,020	3,700	3,020	A-1 & A-2	2015	2015	No	$173.0	Deliveries	99.9	94.4	-	-	Includes G-372 mechanical modification option E to provide continued level of flood control from the Miami Canal
A2-2C	G370 and G 372 modified to pump to 12'	1,500	1,860	3,700	3,360	A-1 & A-2	2015	2015	Yes	$169.2	Deliveries	99.9	94.9	-	-	Includes G-372 mechanical modification option E to provide continued level of flood control from the Miami Canal
A2-2D	G370 and G 372 modified to pump to 12'	1,000	2,220	3,700	3,220	A-1 & A-2	2015	2015	Yes	$164.4	Deliveries	99.9	94.8	-	-	Includes G-372 mechanical modification option E to provide continued level of flood control from the Miami Canal
A2-2E	G370 and G 372 modified to pump to 12'	500	2,775	3,700	3,275	A-1 & A-2	2015	2015	Yes	$147.6	Deliveries	99.9	94.8	-	-	Includes G-372 mechanical modification option E to provide continued level of flood control from the Miami Canal

Figure 6.5-13 Optimization for Deliveries – Alternative A1-1A (G-370 Pump Station with Three Pumps Running)

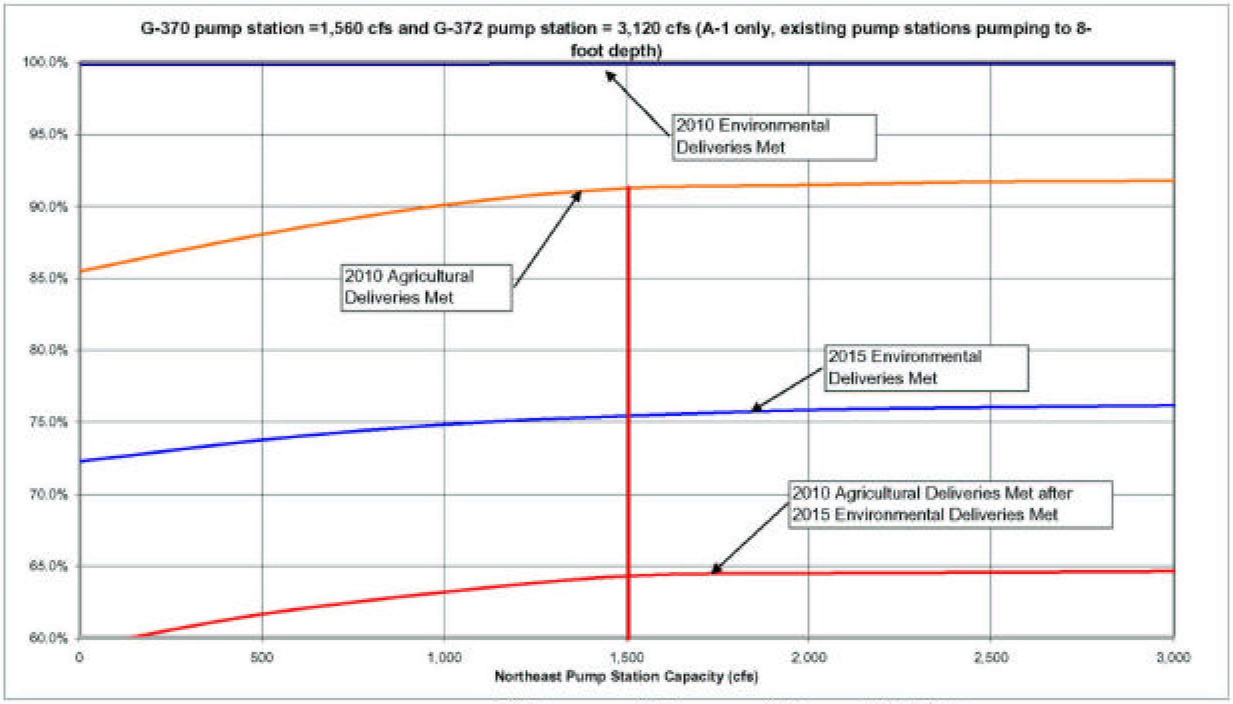


Water Depth (feet)	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	2,340	8	3,120	8	99.7%	85.9%	73.1%	60.7%
12	500	2,340	8	3,120	8	99.7%	88.3%	74.3%	62.4%
12	1,000	2,340	8	3,120	8	99.7%	90.1%	75.0%	63.7%
12	1,500	2,340	8	3,120	8	99.7%	91.3%	75.6%	64.3%
12	2,000	2,340	8	3,120	8	99.7%	91.5%	75.8%	64.4%
12	2,500	2,340	8	3,120	8	99.7%	91.7%	76.0%	64.5%
12	3,000	2,340	8	3,120	8	99.7%	91.7%	76.1%	64.6%
12	3,500	2,340	8	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,000	2,340	8	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,500	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,000	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,500	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,000	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,500	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	7,000	2,340	8	3,120	8	99.7%	91.7%	76.3%	64.6%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,340	8	3,120	8	$ 30,267,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 50,623,000
12	500	2,340	8	3,120	8	$ 37,250,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 60,704,000
12	1,000	2,340	8	3,120	8	$ 44,500,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 67,954,000
12	1,500	2,340	8	3,120	8	$ 50,832,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 86,248,000
12	2,000	2,340	8	3,120	8	$ 56,000,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 91,416,000
12	2,500	2,340	8	3,120	8	$ 60,750,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 96,166,000
12	3,000	2,340	8	3,120	8	$ 64,722,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 109,193,000
12	3,500	2,340	8	3,120	8	$ 80,000,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 130,575,000
12	3,600	2,340	8	3,120	8	$ 84,496,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 135,071,000
12	4,000	2,340	8	3,120	8	$ 87,500,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 138,075,000
12	4,500	2,340	8	3,120	8	$ 91,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 149,356,000
12	5,000	2,340	8	3,120	8	$ 94,647,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 153,003,000
12	5,500	2,340	8	3,120	8	$ 98,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 156,356,000

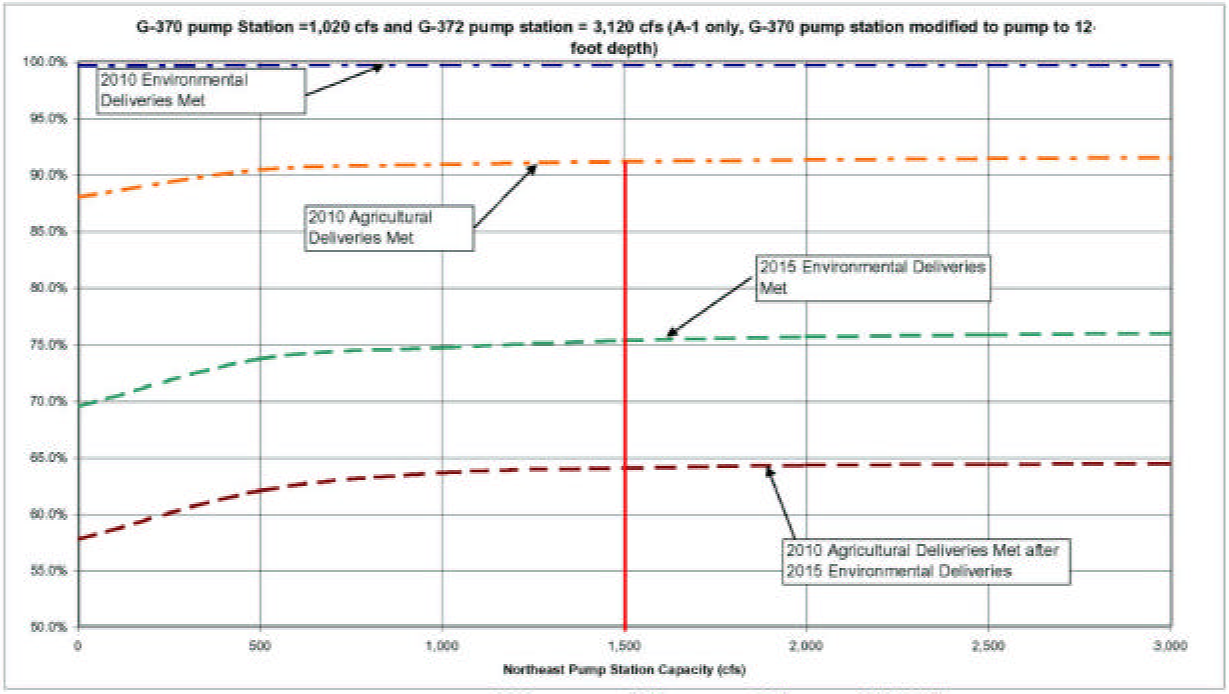
Figure 6.5-14 Optimization for Deliveries – Alternative A1-1A (G-370 Pump Station with Two Pumps Running)



Water Depth (feet)	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	1,560	8	3,120	8	99.7%	85.4%	72.2%	59.3%
12	500	1,560	8	3,120	8	99.7%	87.9%	73.6%	61.5%
12	1,000	1,560	8	3,120	8	99.7%	90.0%	74.7%	63.1%
12	1,500	1,560	8	3,120	8	99.7%	91.1%	75.3%	64.2%
12	2,000	1,560	8	3,120	8	99.7%	91.3%	75.7%	64.4%
12	2,500	1,560	8	3,120	8	99.7%	91.6%	75.9%	64.4%
12	3,000	1,560	8	3,120	8	99.7%	91.6%	76.0%	64.5%
12	3,500	1,560	8	3,120	8	99.7%	91.7%	76.1%	64.6%
12	4,000	1,560	8	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,500	1,560	8	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,000	1,560	8	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,500	1,560	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,000	1,560	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,500	1,560	8	3,120	8	99.7%	91.7%	76.3%	64.6%
12	7,000	1,560	8	3,120	8	99.7%	91.7%	76.3%	64.6%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,560	8	3,120	8	$ 30,267,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 50,623,000
12	500	1,560	8	3,120	8	$ 37,250,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 57,606,000
12	1,000	1,560	8	3,120	8	$ 44,500,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 64,856,000
12	1,500	1,560	8	3,120	8	$ 50,832,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 74,286,000
12	2,000	1,560	8	3,120	8	$ 56,000,000	$ -	$ -	$ 13,856,000	$ 12,540,000	$ 82,396,000
12	2,500	1,560	8	3,120	8	$ 60,750,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 96,166,000
12	3,000	1,560	8	3,120	8	$ 64,722,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 100,138,000
12	3,500	1,560	8	3,120	8	$ 80,000,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 124,471,000
12	3,600	1,560	8	3,120	8	$ 84,496,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 128,967,000
12	4,000	1,560	8	3,120	8	$ 87,500,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 131,971,000
12	4,500	1,560	8	3,120	8	$ 91,000,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 141,575,000
12	5,000	1,560	8	3,120	8	$ 94,647,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 153,003,000
12	5,500	1,560	8	3,120	8	$ 98,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 156,356,000

Figure 6.5-15 Optimization for Deliveries – Alternative A1-2B

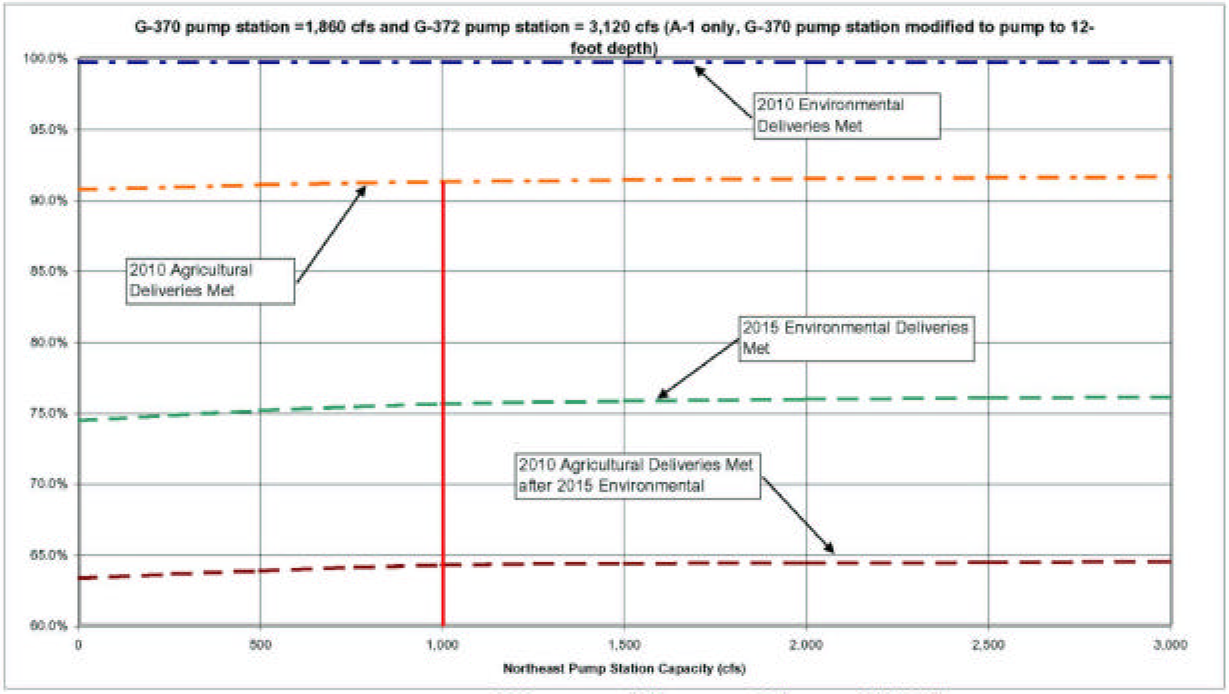


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	1,020	12	3,120	8	99.7%	88.1%	69.5%	57.8%
12	500	1,020	12	3,120	8	99.8%	90.5%	73.8%	62.1%
12	1,000	1,020	12	3,120	8	99.7%	91.0%	74.8%	63.7%
12	1,500	1,020	12	3,120	8	99.7%	91.2%	75.4%	64.1%
12	2,000	1,020	12	3,120	8	99.7%	91.4%	75.7%	64.4%
12	2,500	1,020	12	3,120	8	99.7%	91.5%	75.9%	64.4%
12	3,000	1,020	12	3,120	8	99.7%	91.6%	76.0%	64.5%
12	3,500	1,020	12	3,120	8	99.7%	91.6%	76.1%	64.5%
12	4,000	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,500	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,000	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,500	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	6,000	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	6,500	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	7,000	1,020	12	3,120	8	99.7%	91.7%	76.2%	64.6%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,020	12	3,120	8	$ 30,267,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 82,443,000
12	500	1,020	12	3,120	8	$ 37,250,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 89,426,000
12	1,000	1,020	12	3,120	8	$ 44,500,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 96,676,000
12	1,500	1,020	12	3,120	8	$ 50,832,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 103,008,000
12	2,000	1,020	12	3,120	8	$ 56,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 9,598,000	$ 111,274,000
12	2,500	1,020	12	3,120	8	$ 60,750,000	$ 3,600,000	$ -	$ 42,076,000	$ 12,540,000	$ 118,966,000
12	3,000	1,020	12	3,120	8	$ 64,722,000	$ 3,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 131,958,000
12	3,500	1,020	12	3,120	8	$ 80,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 147,236,000
12	4,000	1,020	12	3,120	8	$ 87,500,000	$ 3,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 163,791,000
12	4,500	1,020	12	3,120	8	$ 91,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 167,291,000
12	5,000	1,020	12	3,120	8	$ 94,647,000	$ 3,600,000	$ -	$ 42,076,000	$ 36,719,000	$ 177,042,000
12	5,500	1,020	12	3,120	8	$ 98,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 188,176,000
12	6,000	1,020	12	3,120	8	$ 101,750,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 191,926,000
12	6,500	1,020	12	3,120	8	$ 105,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 195,176,000

Figure 6.5-16 Optimization for Deliveries – Alternative A1-2C

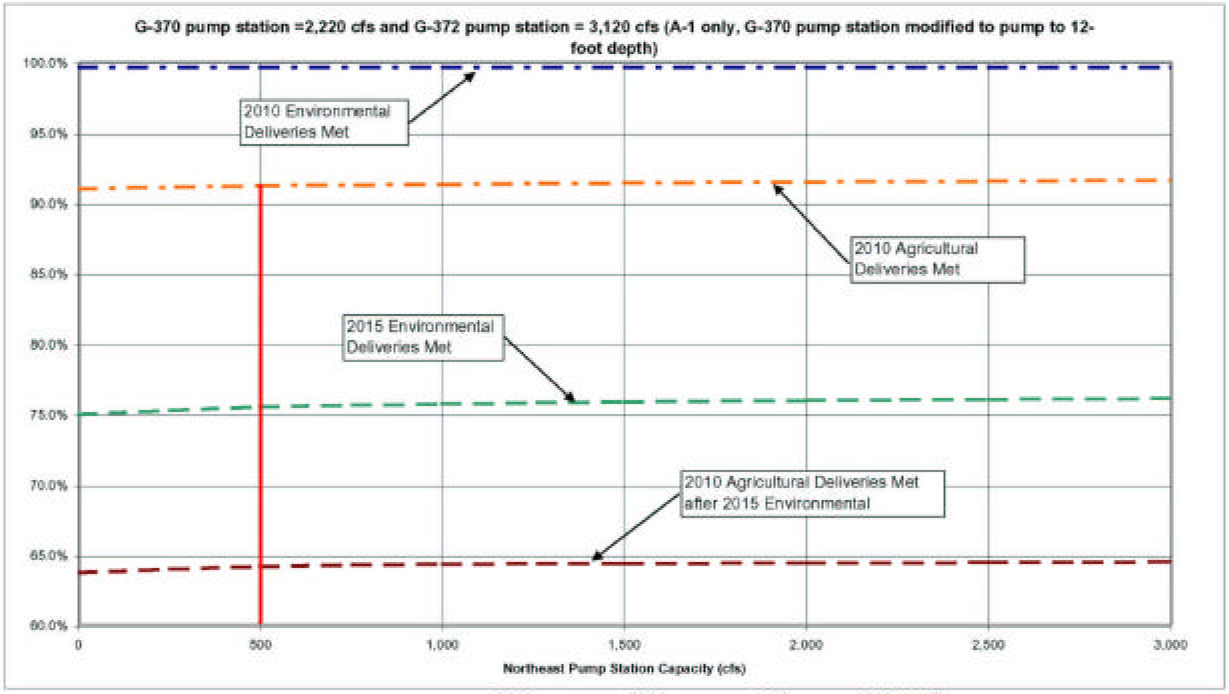


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	1,860	12	3,120	8	99.7%	90.8%	74.5%	63.4%
12	500	1,860	12	3,120	8	99.7%	91.1%	75.2%	63.9%
12	1,000	1,860	12	3,120	8	99.7%	91.3%	75.7%	64.3%
12	1,500	1,860	12	3,120	8	99.7%	91.5%	75.9%	64.4%
12	2,000	1,860	12	3,120	8	99.7%	91.6%	76.0%	64.5%
12	2,500	1,860	12	3,120	8	99.7%	91.6%	76.1%	64.5%
12	3,000	1,860	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	3,500	1,860	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,000	1,860	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,500	1,860	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,000	1,860	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,500	1,860	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,000	1,860	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,500	1,860	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	7,000	1,860	12	3,120	8	99.7%	91.7%	76.2%	64.6%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,860	12	3,120	8	$ 30,267,000	$ 10,200,000	$ -	$ 42,076,000	$ 6,500,000	$ 89,043,000
12	500	1,860	12	3,120	8	$ 37,250,000	$ 10,200,000	$ -	$ 42,076,000	$ 6,500,000	$ 96,026,000
12	1,000	1,860	12	3,120	8	$ 44,500,000	$ 10,200,000	$ -	$ 42,076,000	$ 9,598,000	$ 106,374,000
12	1,500	1,860	12	3,120	8	$ 50,832,000	$ 10,200,000	$ -	$ 42,076,000	$ 9,598,000	$ 112,706,000
12	2,000	1,860	12	3,120	8	$ 56,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 129,836,000
12	2,500	1,860	12	3,120	8	$ 60,750,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 134,586,000
12	3,000	1,860	12	3,120	8	$ 64,722,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 138,558,000
12	3,500	1,860	12	3,120	8	$ 80,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 30,615,000	$ 162,891,000
12	4,000	1,860	12	3,120	8	$ 87,500,000	$ 10,200,000	$ -	$ 42,076,000	$ 36,719,000	$ 176,495,000
12	4,500	1,860	12	3,120	8	$ 91,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 36,719,000	$ 179,995,000
12	5,000	1,860	12	3,120	8	$ 94,647,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 191,423,000
12	5,500	1,860	12	3,120	8	$ 98,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 194,776,000
12	6,000	1,860	12	3,120	8	$ 101,750,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 198,526,000
12	6,500	1,860	12	3,120	8	$ 105,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 201,776,000

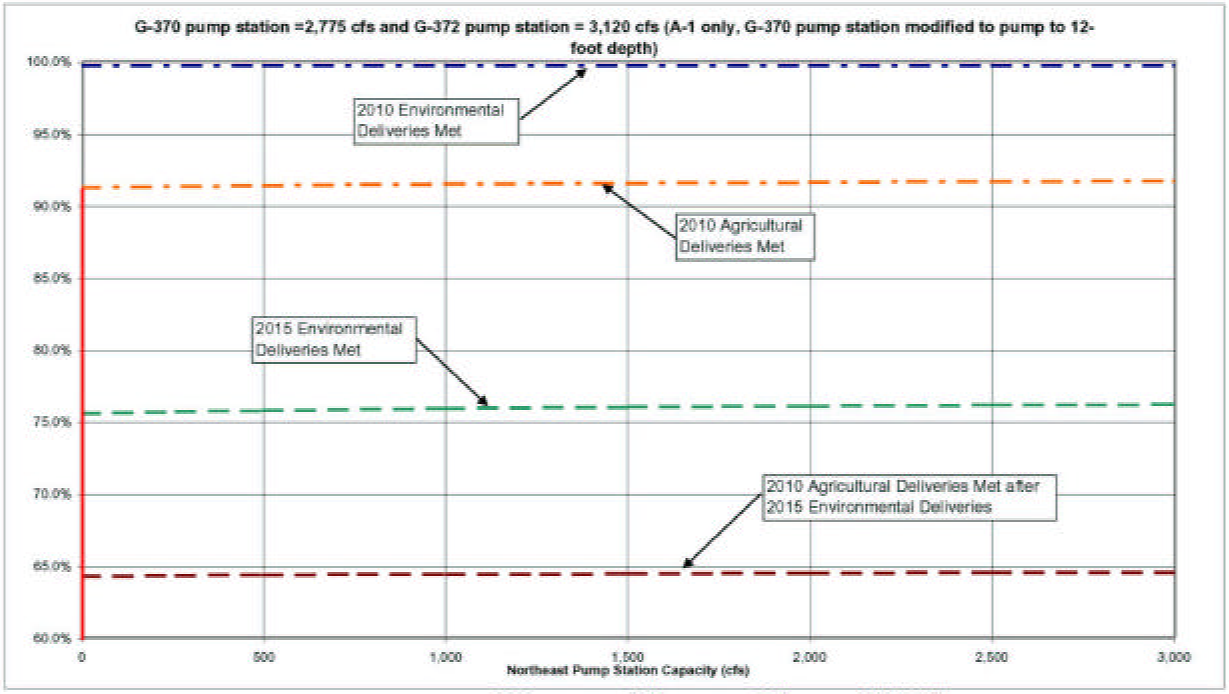
Figure 6.5-17 Optimization for Deliveries – Alternative A1-2D



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	2,220	12	3,120	8	99.7%	91.1%	75.1%	63.8%
12	500	2,220	12	3,120	8	99.7%	91.3%	75.6%	64.3%
12	1,000	2,220	12	3,120	8	99.7%	91.4%	75.8%	64.4%
12	1,500	2,220	12	3,120	8	99.7%	91.5%	76.0%	64.5%
12	2,000	2,220	12	3,120	8	99.7%	91.6%	76.1%	64.5%
12	2,500	2,220	12	3,120	8	99.7%	91.7%	76.1%	64.5%
12	3,000	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	3,500	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,000	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,500	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	5,000	2,220	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,500	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	6,000	2,220	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,500	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	7,000	2,220	12	3,120	8	99.7%	91.7%	76.2%	64.6%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,220	12	3,120	8	$ 30,267,000	$ 11,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 90,443,000
12	500	2,220	12	3,120	8	$ 37,250,000	$ 11,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 97,426,000
12	1,000	2,220	12	3,120	8	$ 44,500,000	$ 11,600,000	$ -	$ 42,076,000	$ 9,598,000	$ 107,774,000
12	1,500	2,220	12	3,120	8	$ 50,832,000	$ 11,600,000	$ -	$ 42,076,000	$ 12,540,000	$ 117,048,000
12	2,000	2,220	12	3,120	8	$ 56,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 131,236,000
12	2,500	2,220	12	3,120	8	$ 60,750,000	$ 11,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 135,986,000
12	3,000	2,220	12	3,120	8	$ 64,722,000	$ 11,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 149,013,000
12	3,500	2,220	12	3,120	8	$ 80,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 164,291,000
12	4,000	2,220	12	3,120	8	$ 87,500,000	$ 11,600,000	$ -	$ 42,076,000	$ 36,719,000	$ 177,895,000
12	4,500	2,220	12	3,120	8	$ 91,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 189,176,000
12	5,000	2,220	12	3,120	8	$ 94,647,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 192,823,000
12	5,500	2,220	12	3,120	8	$ 98,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 196,176,000
12	6,000	2,220	12	3,120	8	$ 101,750,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 199,926,000
12	6,500	2,220	12	3,120	8	$ 105,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 203,176,000

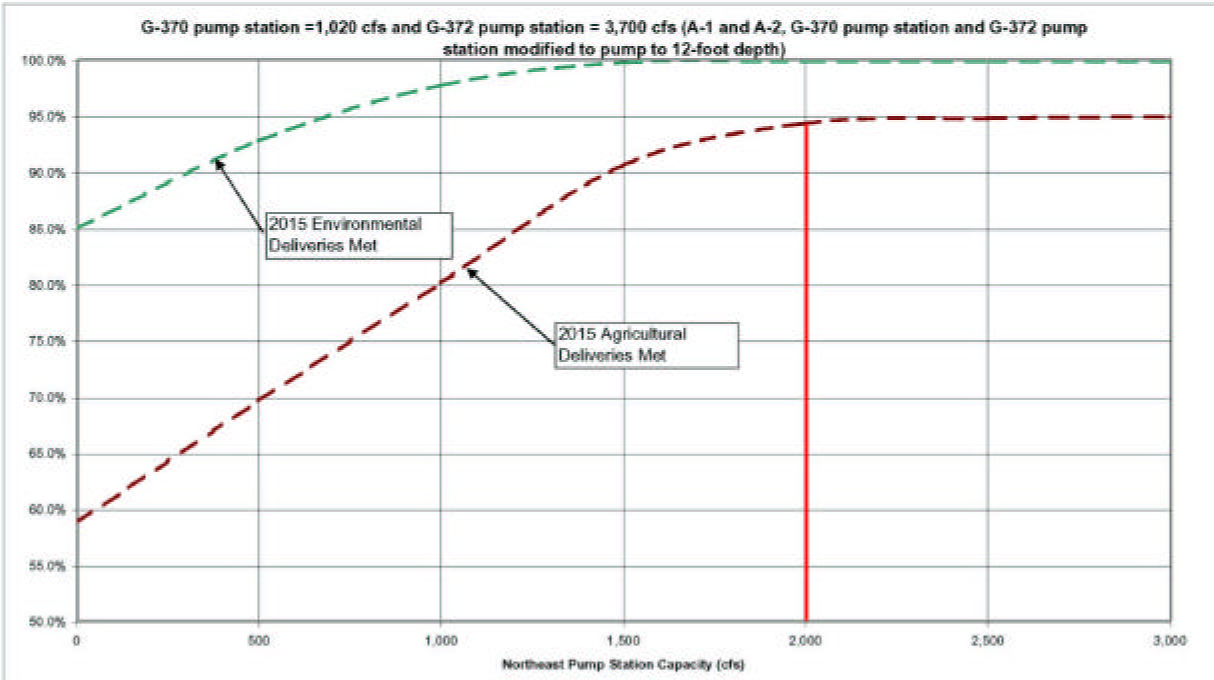
Figure 6.5-18 Optimization for Deliveries – Alternative A1-2E



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2010 Percentage of Envtl Deliveries Met	2010 Percentage of Agricultural Deliveries Met	2015 Percentage of Envtl Deliveries Met	2010 (2015) Percentage of Agricultural Deliveries Met
12	0	2,775	12	3,120	8	99.7%	91.3%	75.6%	64.3%
12	500	2,775	12	3,120	8	99.7%	91.4%	75.8%	64.4%
12	1,000	2,775	12	3,120	8	99.7%	91.5%	76.0%	64.5%
12	1,500	2,775	12	3,120	8	99.7%	91.6%	76.1%	64.5%
12	2,000	2,775	12	3,120	8	99.7%	91.7%	76.1%	64.5%
12	2,500	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	3,000	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	3,500	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	4,000	2,775	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	4,500	2,775	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,000	2,775	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	5,500	2,775	12	3,120	8	99.7%	91.7%	76.3%	64.6%
12	6,000	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	6,500	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%
12	7,000	2,775	12	3,120	8	99.7%	91.7%	76.2%	64.6%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,775	12	3,120	8	$ 30,267,000	$ 13,100,000	$ -	$ 42,076,000	$ 6,500,000	$ 91,943,000
12	500	2,775	12	3,120	8	$ 37,250,000	$ 13,100,000	$ -	$ 42,076,000	$ 9,598,000	$ 102,024,000
12	1,000	2,775	12	3,120	8	$ 44,500,000	$ 13,100,000	$ -	$ 42,076,000	$ 12,540,000	$ 112,216,000
12	1,500	2,775	12	3,120	8	$ 50,832,000	$ 13,100,000	$ -	$ 42,076,000	$ 21,560,000	$ 127,568,000
12	2,000	2,775	12	3,120	8	$ 56,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 21,560,000	$ 132,736,000
12	2,500	2,775	12	3,120	8	$ 60,750,000	$ 13,100,000	$ -	$ 42,076,000	$ 30,615,000	$ 146,541,000
12	3,000	2,775	12	3,120	8	$ 64,722,000	$ 13,100,000	$ -	$ 42,076,000	$ 30,615,000	$ 150,513,000
12	3,500	2,775	12	3,120	8	$ 80,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 36,719,000	$ 171,895,000
12	4,000	2,775	12	3,120	8	$ 87,500,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 187,176,000
12	4,500	2,775	12	3,120	8	$ 91,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 190,676,000
12	5,000	2,775	12	3,120	8	$ 94,647,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 194,323,000
12	5,500	2,775	12	3,120	8	$ 98,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 197,676,000
12	6,000	2,775	12	3,120	8	$ 101,750,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 201,426,000
12	6,500	2,775	12	3,120	8	$ 105,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 204,676,000

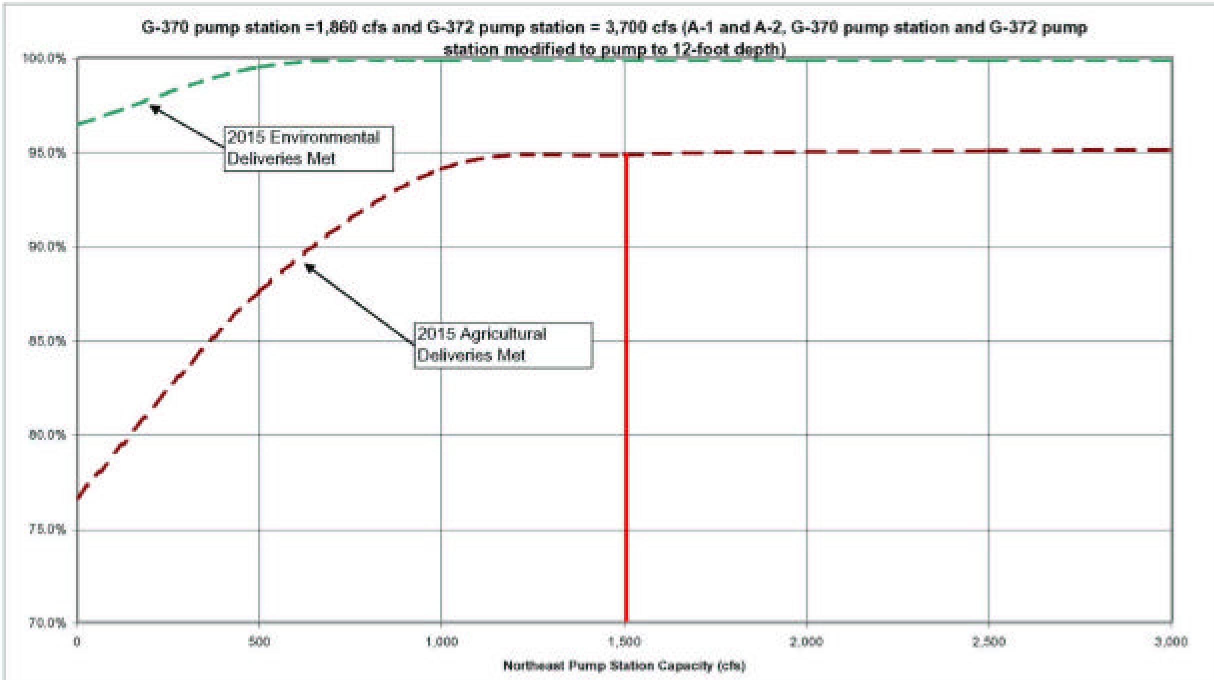
Figure 6.5-19 Optimization for Deliveries – Alternative A2-2B



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Envtl Deliveries Met	2015 Percentage of Agricultural Deliveries Met
12	0	1,020	12	3,700	12	85.1%	58.9%
12	500	1,020	12	3,700	12	92.9%	69.8%
12	1,000	1,020	12	3,700	12	97.8%	80.3%
12	1,500	1,020	12	3,700	12	99.8%	90.7%
12	2,000	1,020	12	3,700	12	99.9%	94.4%
12	2,500	1,020	12	3,700	12	99.9%	94.9%
12	3,000	1,020	12	3,700	12	99.9%	95.0%
12	3,500	1,020	12	3,700	12	99.9%	95.1%
12	4,000	1,020	12	3,700	12	99.9%	95.1%
12	4,500	1,020	12	3,700	12	99.9%	95.1%
12	5,000	1,020	12	3,700	12	99.9%	95.1%
12	5,500	1,020	12	3,700	12	99.9%	95.1%
12	6,000	1,020	12	3,700	12	99.9%	95.1%
12	6,500	1,020	12	3,700	12	99.9%	95.1%
12	7,000	1,020	12	3,700	12	99.9%	95.1%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,020	12	3,700	12	$ 30,267,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 144,155,000
12	500	1,020	12	3,700	12	$ 37,250,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 151,138,000
12	1,000	1,020	12	3,700	12	$ 44,500,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 158,388,000
12	1,500	1,020	12	3,700	12	$ 50,832,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 164,720,000
12	2,000	1,020	12	3,700	12	$ 56,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 172,986,000
12	2,500	1,020	12	3,700	12	$ 60,750,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 180,678,000
12	3,000	1,020	12	3,700	12	$ 64,722,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 193,670,000
12	3,500	1,020	12	3,700	12	$ 80,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 208,948,000
12	4,000	1,020	12	3,700	12	$ 87,500,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 225,503,000
12	4,500	1,020	12	3,700	12	$ 91,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 229,003,000
12	5,000	1,020	12	3,700	12	$ 94,647,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 238,754,000
12	5,500	1,020	12	3,700	12	$ 98,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 249,888,000
12	6,000	1,020	12	3,700	12	$ 101,750,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 253,638,000
12	6,500	1,020	12	3,700	12	$ 105,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 256,888,000

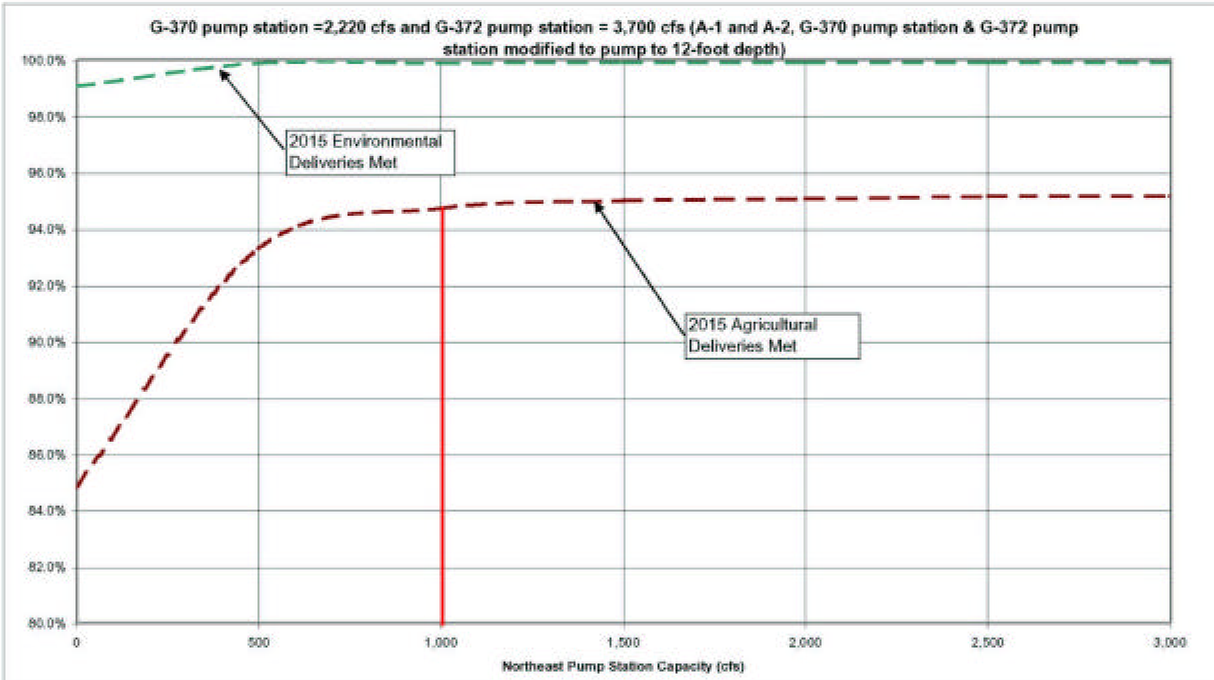
Figure 6.5-20 Optimization for Deliveries – Alternative A2-2C



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Envtl Deliveries Met	2015 Percentage of Agricultural Deliveries Met
12	0	1,860	12	3,700	12	96.5%	76.7%
12	500	1,860	12	3,700	12	99.6%	87.6%
12	1,000	1,860	12	3,700	12	99.9%	94.2%
12	1,500	1,860	12	3,700	12	99.9%	94.9%
12	2,000	1,860	12	3,700	12	99.9%	95.1%
12	2,500	1,860	12	3,700	12	99.9%	95.1%
12	3,000	1,860	12	3,700	12	99.9%	95.2%
12	3,500	1,860	12	3,700	12	99.9%	95.2%
12	4,000	1,860	12	3,700	12	99.9%	95.2%
12	4,500	1,860	12	3,700	12	99.9%	95.2%
12	5,000	1,860	12	3,700	12	99.9%	95.2%
12	5,500	1,860	12	3,700	12	99.9%	95.2%
12	6,000	1,860	12	3,700	12	99.9%	95.2%
12	6,500	1,860	12	3,700	12	99.9%	95.2%
12	7,000	1,860	12	3,700	12	99.9%	95.2%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,860	12	3,700	12	$ 30,267,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 145,555,000
12	500	1,860	12	3,700	12	$ 37,250,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 152,538,000
12	1,000	1,860	12	3,700	12	$ 44,500,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 162,886,000
12	1,500	1,860	12	3,700	12	$ 50,832,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 169,218,000
12	2,000	1,860	12	3,700	12	$ 56,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 186,348,000
12	2,500	1,860	12	3,700	12	$ 60,750,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 191,098,000
12	3,000	1,860	12	3,700	12	$ 64,722,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 195,070,000
12	3,500	1,860	12	3,700	12	$ 80,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 219,403,000
12	4,000	1,860	12	3,700	12	$ 87,500,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 233,007,000
12	4,500	1,860	12	3,700	12	$ 91,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 236,507,000
12	5,000	1,860	12	3,700	12	$ 94,647,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 247,935,000
12	5,500	1,860	12	3,700	12	$ 98,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 251,288,000
12	6,000	1,860	12	3,700	12	$ 101,750,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 255,038,000
12	6,500	1,860	12	3,700	12	$ 105,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 258,288,000

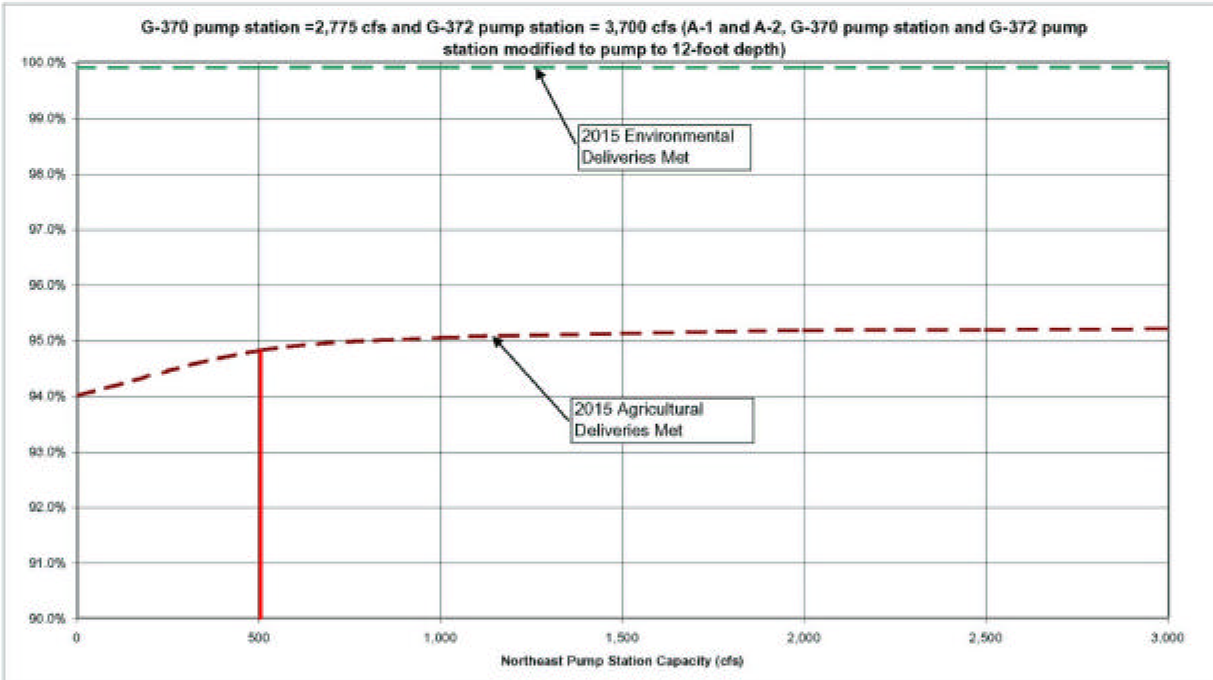
Figure 6.5-21 Optimization for Deliveries – Alternative A2-2D



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Envtl Deliveries Met	2015 Percentage of Agricultural Deliveries Met
12	0	2,220	12	3,700	12	99.1%	84.9%
12	500	2,220	12	3,700	12	99.9%	93.3%
12	1,000	2,220	12	3,700	12	99.9%	94.8%
12	1,500	2,220	12	3,700	12	99.9%	95.0%
12	2,000	2,220	12	3,700	12	99.9%	95.1%
12	2,500	2,220	12	3,700	12	99.9%	95.2%
12	3,000	2,220	12	3,700	12	99.9%	95.2%
12	3,500	2,220	12	3,700	12	99.9%	95.2%
12	4,000	2,220	12	3,700	12	99.9%	95.2%
12	4,500	2,220	12	3,700	12	99.9%	95.2%
12	5,000	2,220	12	3,700	12	99.9%	95.2%
12	5,500	2,220	12	3,700	12	99.9%	95.2%
12	6,000	2,220	12	3,700	12	99.9%	95.2%
12	6,500	2,220	12	3,700	12	99.9%	95.2%
12	7,000	2,220	12	3,700	12	99.9%	95.2%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,220	12	3,700	12	$ 30,267,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 147,055,000
12	500	2,220	12	3,700	12	$ 37,250,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 154,038,000
12	1,000	2,220	12	3,700	12	$ 44,500,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 164,386,000
12	1,500	2,220	12	3,700	12	$ 50,832,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 173,660,000
12	2,000	2,220	12	3,700	12	$ 56,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 187,848,000
12	2,500	2,220	12	3,700	12	$ 60,750,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 192,598,000
12	3,000	2,220	12	3,700	12	$ 64,722,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 205,625,000
12	3,500	2,220	12	3,700	12	$ 80,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 220,903,000
12	4,000	2,220	12	3,700	12	$ 87,500,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 234,507,000
12	4,500	2,220	12	3,700	12	$ 91,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 245,788,000
12	5,000	2,220	12	3,700	12	$ 94,647,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 249,435,000
12	5,500	2,220	12	3,700	12	$ 98,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 252,788,000
12	6,000	2,220	12	3,700	12	$ 101,750,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 256,538,000
12	6,500	2,220	12	3,700	12	$ 105,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 259,788,000

Figure 6.5-22 Optimization for Deliveries – Alternative A2-2E



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Envtl Deliveries Met	2015 Percentage of Agricultural Deliveries Met
12	0	2,775	12	3,700	12	99.9%	94.0%
12	500	2,775	12	3,700	12	99.9%	94.8%
12	1,000	2,775	12	3,700	12	99.9%	95.1%
12	1,500	2,775	12	3,700	12	99.9%	95.1%
12	2,000	2,775	12	3,700	12	99.9%	95.2%
12	2,500	2,775	12	3,700	12	99.9%	95.2%
12	3,000	2,775	12	3,700	12	99.9%	95.2%
12	3,500	2,775	12	3,700	12	99.9%	95.2%
12	4,000	2,775	12	3,700	12	99.9%	95.2%
12	4,500	2,775	12	3,700	12	99.9%	95.2%
12	5,000	2,775	12	3,700	12	99.9%	95.2%
12	5,500	2,775	12	3,700	12	99.9%	95.2%
12	6,000	2,775	12	3,700	12	99.9%	95.2%
12	6,500	2,775	12	3,700	12	99.9%	95.2%
12	7,000	2,775	12	3,700	12	99.9%	95.2%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,775	12	3,700	12	$ 30,267,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 137,555,000
12	500	2,775	12	3,700	12	$ 37,250,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 147,636,000
12	1,000	2,775	12	3,700	12	$ 44,500,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 157,828,000
12	1,500	2,775	12	3,700	12	$ 50,832,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 173,180,000
12	2,000	2,775	12	3,700	12	$ 56,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 178,348,000
12	2,500	2,775	12	3,700	12	$ 60,750,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 192,153,000
12	3,000	2,775	12	3,700	12	$ 64,722,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 196,125,000
12	3,500	2,775	12	3,700	12	$ 80,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 217,507,000
12	4,000	2,775	12	3,700	12	$ 87,500,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 232,788,000
12	4,500	2,775	12	3,700	12	$ 91,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 236,288,000
12	5,000	2,775	12	3,700	12	$ 94,647,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 239,935,000
12	5,500	2,775	12	3,700	12	$ 98,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 243,288,000
12	6,000	2,775	12	3,700	12	$ 101,750,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 247,038,000
12	6,500	2,775	12	3,700	12	$ 105,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 250,288,000

6.5.2.4 Optimization for Priority Removals

Another goal identified by SFWMD is to maximize priority removals by providing sufficient EAA Reservoir A-1, pumping station, and canal capacity. Priority removals are defined as follows (in order of declining preference):

- Local runoff within the NNRC and Miami Canal drainage area
- Pump backs to Lake Okeechobee from the NNRC by S-2 pump station and from the Miami Canal by S-3 pump station
- Regulatory releases from Lake Okeechobee

Unlike the optimization described above for water deliveries for which the minimum necessary pump station capacity is the desirable result, this optimization step requires the identification of the largest practical pump station size that will cost effectively respond to stormwater runoff. Application of the Water Balance Model demonstrated that, for both the 2010 and the 2015 water availability and delivery conditions, there is sufficient water available from the Lake Okeechobee regulatory releases to meet deliveries, so much in fact that the EAA Reservoir A-1 could be maintained in a relatively full state for all but the driest climate conditions. While this may seem desirable from a delivery standpoint, in order to improve the preferred priority removals of local runoff and pump backs, the water level in the EAA Reservoir A-1 will need to be carefully managed to ensure that sufficient storage capacity is available to allow discharge into the EAA Reservoir A-1 at any time when runoff conditions occur. This will mean that Lake Okeechobee regulatory releases into the EAA Reservoir A-1 may be deferred at times in favor of maintaining operational capacity for the local runoff and the pump backs. Therefore, in conducting this evaluation, the Lake Okeechobee regulatory releases were deleted from the model runs so that maximum storage capacity was made available for local runoff and pump backs, and the associated pump station capacity required to capture these flows could be identified. Actual operation will include Lake Okeechobee regulatory releases, but in a managed quantity and duration.

Figures 6.5-23 through 6.5-31 provide individual summaries of each alternative evaluated for this refinement. The summaries identify the percent of local runoff that can be captured, the percent of pump backs that can be routed to the new EAA Reservoir A-1, and the costs associated with each. The alternatives are also summarized in Table 6.5-7 and are based on the following:

- For the EAA Reservoir A-1, a new northeast pump station in combination with G-370 and G-372 pump stations in an unmodified state, pumping into the EAA Reservoir A-1 when water depths are less than eight feet and into the STA-3/4 Supply Canal when EAA Reservoir A-1 water depths are greater than eight feet.
- For the EAA Reservoir A-1, a new northeast pump station in combination with G-370 pump station modified to pump to full EAA Reservoir A-1 depth and G-372 pump station in an unmodified state pumping into the EAA Reservoir A-1 when water depths are less than eight feet and into the STA-3/4 Supply Canal when EAA Reservoir A-1 water depths are greater than eight feet.

- For the EAA Reservoir A1 and EAA Reservoir A-2, a new northeast pump station in combination with G-370 and G-372 pump stations modified to pump to full EAA Reservoir A-1 depth.

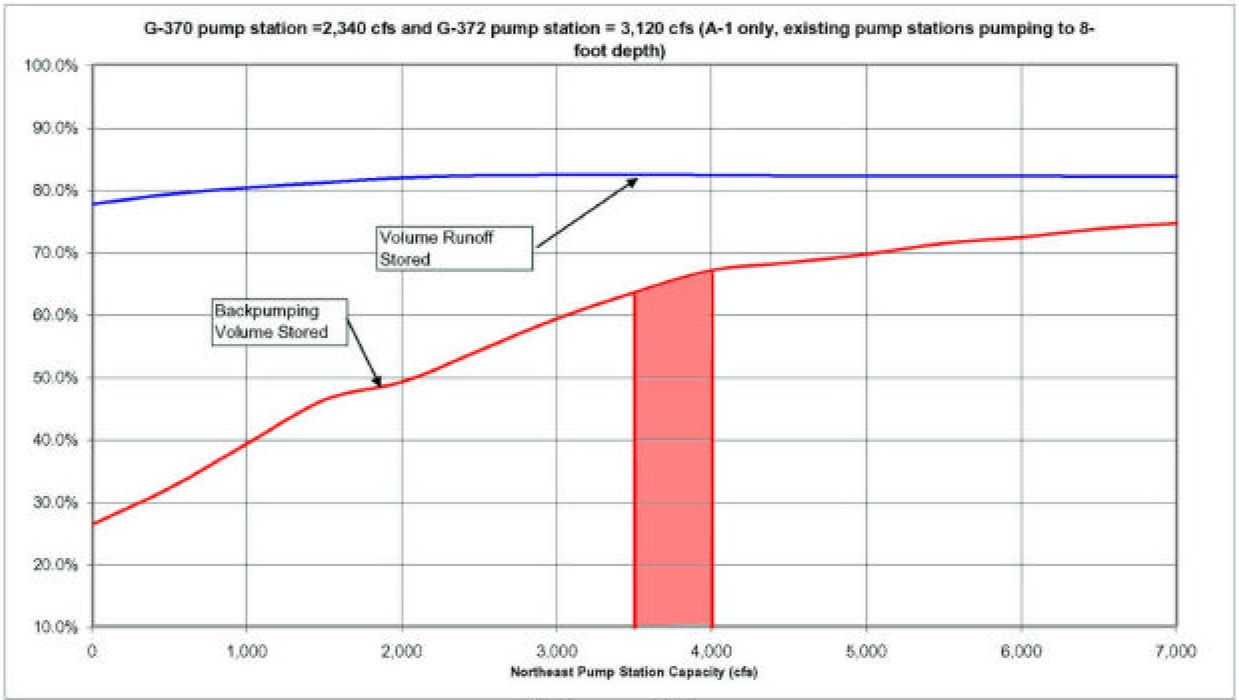
As with the previous refinement, for the first phase of EAA Reservoir A-1 construction it was assumed that there would be no modification to G-372 pump station due to the associated high cost identified during the preliminary screening. Consequently, this BODR focuses on reduction of priority removals in the NNRC drainage area. From Table 6.5-7 it appears that a combined capacity for the northeast pump station and G-370 pump station in the range of about 6,000 cfs would be sufficient to maximize the amount of local runoff and pump backs that could be captured in the EAA Reservoir A-1. The graph on Figure 6.5-23 shows a definite break point for a northeast pump station capacity of about 4,000 cfs. Evaluations employing various adjustments to the WBM all resulted in northeast pump station capacities ranging from 3,500 to 4,000 cfs. Additional removals could be achieved with greater pump station capacities, but at greater cost and steadily diminishing returns. Modifications would be required to the NNRC to provide the greater conveyance capacity needed to allow pumping at this rate.

For the second phase of the EAA Reservoir A-1 construction, the change in the graph's slope is less defined, but it appears that a combined capacity between the northeast pump station and G-370 pump station in the range of 5,000 to 6,000 cfs would sufficiently maximize the amount of local runoff and pump backs that could be captured in the combined EAA Reservoir A-1 and A-2. Alternative A2-2B matches the capacity for the northeast pump station identified above for phase one with one of the modification options for G-370 pump station. No additional canal modifications beyond those identified for phase one would be needed with this alternative.

Table 6.5-7 Costs Associated with Pump Station Alternatives and Options Based on Priority Removals

		Capacity														
Option	Description	Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Combined Capacity - Northeast Pump Station and G-370 Pump Station	EAA Reservoir	Flow Condition	Delivery Condition	Canal Improvement	Cost ($Millions)	Optimization Goal	Envt'l Delivery % Met	Irrigation Delivery % Met	Runoff Captured % Stored	Back Pumping % Captured	Comment
A1-1A	G370 and G372 unmodified	3,600	2,340	3,120	5,940	A-1	2010	2015	Yes	$135.1	Priority Removals	-	-	82.3	63.4	Delivery percentage for both environmental and irrigation is maxed out; increased NE capacity would not improve. Runoff stored is maxed out; increased NE capacity would not improve. Increases in pump back reduction reaches a breakpoint; further increases in NE capacity would further reduce pumpbacks, but with diminishing returns.
A1-2B	G-370 pump station modified to 12', G-372 pump station unmodified	3,000	1,020	3,120	4,020	A-1	2010	2010 & 2015	No	$131.2	Priority Removals	-	-	76.7	58.8	Although lower cost than Option A1-1A, provides lower priority removal and flood control from the NNRC than that option
A1-2C	G-370 pump station modified to 12', G-372 pump station unmodified	2,000	1,860	3,120	3,860	A-1	2010	2010 & 2015	No	$129.8	Priority Removals	-	-	76.7	57.3	Although lower cost than Option A1-1A, provides lower priority removal and flood control from the NNRC than that option
A1-2D	G-370 pump station modified to 12', G-372 pump station unmodified	1,500	2,220	3,120	3,720	A-1	2010	2010 & 2015	No	$117.0	Priority Removals	-	-	76.7	56.2	Although lower cost than Option A1-1A, provides lower priority removal and flood control from the NNRC than that option
A1-2E	G-370 pump station modified to 12', G-372 pump station unmodified	1,000	2,775	3,120	3,775	A-1	2010	2010 & 2015	Yes	$112.2	Priority Removals	-	-	76.7	56.4	Although lower cost than Option A1-1A, provides lower priority removal and flood control from the NNRC than that option
A2-2B	G370 and G 372 modified to pump to 12'	5,000	1,020	3,700	6,020	A-1 & A-2	2015	2015	Yes	$208.5	Priority Removals	-	-	95.4	76.5	Delivery percentage for both environmental and irrigation is maxed out; increased NE capacity would not improve. Runoff stored is maxed out; increased NE capacity would not improve. Increases in pump back reduction reaches a breakpoint; further increases in NE capacity would further reduce pumpbacks, but with diminishing returns.
A2-2C	G370 and G 372 modified to pump to 12'	4,000	1,860	3,700	5,860	A-1 & A-2	2015	2015	Yes	$202.8	Priority Removals	-	-	95.3	76.7	Delivery percentage for both environmental and irrigation is maxed out; increased NE capacity would not improve. Runoff stored is maxed out; increased NE capacity would not improve. Increases in pump back reduction reaches a breakpoint; further increases in NE capacity would further reduce pumpbacks, but with diminishing returns.
2A-2D	G370 and G 372 modified to pump to 12'	3,600	2,220	3,700	5,820	A-1 & A-2	2015	2015	Yes	$196.8	Priority Removals	-	-	95.3	75.8	Delivery percentage for both environmental and irrigation is maxed out; increased NE capacity would not improve. Runoff stored is maxed out; increased NE capacity would not improve. Increases in pump back reduction reaches a breakpoint; further increases in NE capacity would further reduce pumpbacks, but with diminishing returns. No significant advantage over Option A-4
2A-2E	G370 and G 372 modified to pump to 12'	3,000	2,775	3,700	5,775	A-1 & A-2	2015	2015	Yes	$172.0	Priority Removals	-	-	95.3	76.2	Delivery percentage for both environmental and irrigation is maxed out; increased NE capacity would not improve. Runoff stored is maxed out; increased NE capacity would not improve. Increases in pump back reduction reaches a breakpoint; further increases in NE capacity would further reduce pumpbacks, but with diminishing returns. No significant advantage over Option A-4

Figure 6.5-23 Optimization for Priority Removals – Alternative A1-1A (G-370 Pump Station with Three Pumps Running)

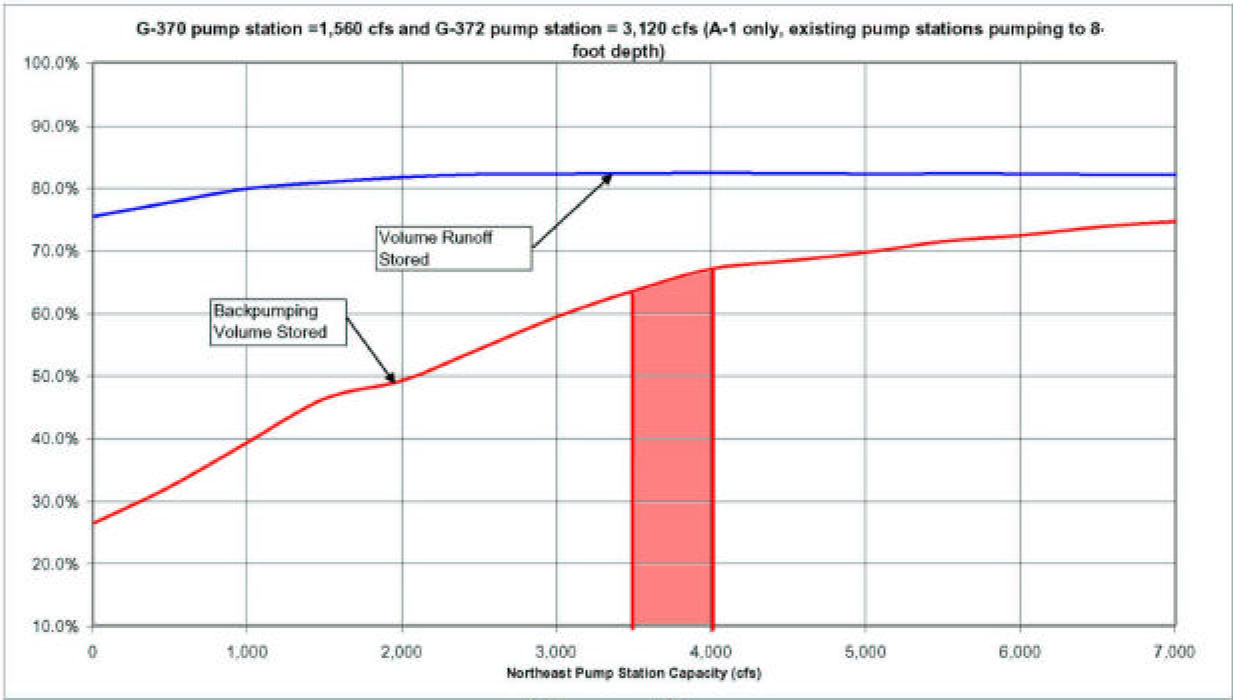


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	2,340	8	3,120	8	77.6%	26.3%
12	500	2,340	8	3,120	8	79.1%	32.2%
12	1,000	2,340	8	3,120	8	80.2%	39.2%
12	1,500	2,340	8	3,120	8	81.0%	46.2%
12	2,000	2,340	8	3,120	8	81.8%	49.1%
12	2,500	2,340	8	3,120	8	82.2%	54.2%
12	3,000	2,340	8	3,120	8	82.3%	59.2%
12	3,500	2,340	8	3,120	8	82.3%	63.4%
12	4,000	2,340	8	3,120	8	82.2%	66.9%
12	4,500	2,340	8	3,120	8	82.1%	68.2%
12	5,000	2,340	8	3,120	8	82.1%	69.5%
12	5,500	2,340	8	3,120	8	82.1%	71.3%
12	6,000	2,340	8	3,120	8	82.0%	72.2%
12	6,500	2,340	8	3,120	8	82.0%	73.6%
12	7,000	2,340	8	3,120	8	82.0%	74.5%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,340	8	3,120	8	$ 30,267,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 50,623,000
12	500	2,340	8	3,120	8	$ 37,250,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 60,704,000
12	1,000	2,340	8	3,120	8	$ 44,500,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 67,954,000
12	1,500	2,340	8	3,120	8	$ 50,832,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 86,248,000
12	2,000	2,340	8	3,120	8	$ 56,000,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 91,416,000
12	2,500	2,340	8	3,120	8	$ 60,750,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 96,166,000
12	3,000	2,340	8	3,120	8	$ 64,722,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 109,193,000
12	3,500	2,340	8	3,120	8	$ 80,000,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 130,575,000
12	3,600	2,340	8	3,120	8	$ 84,496,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 135,071,000
12	4,000	2,340	8	3,120	8	$ 87,500,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 138,075,000
12	4,500	2,340	8	3,120	8	$ 91,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 149,356,000
12	5,000	2,340	8	3,120	8	$ 94,647,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 153,003,000
12	5,500	2,340	8	3,120	8	$ 98,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 156,356,000

Figure 6.5-24 Optimization for Priority Removals – Alternative A1-1A (G-370 Pump Station with Two Pumps Running)

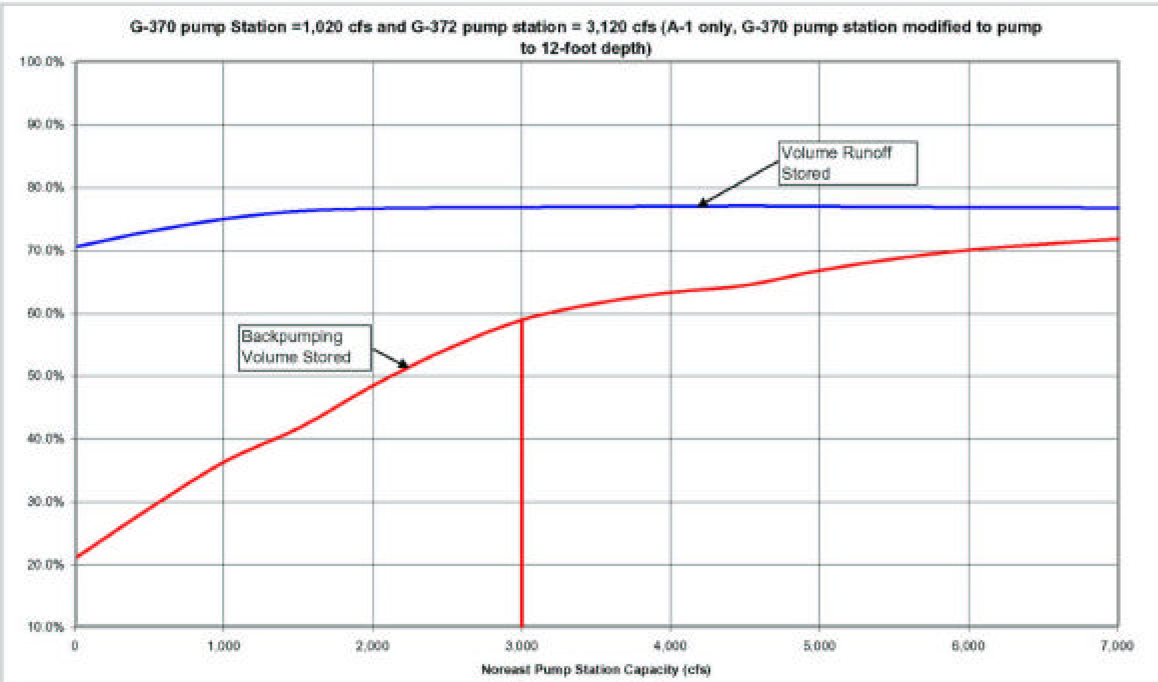


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	1,560	8	3,120	8	75.3%	22.7%
12	500	1,560	8	3,120	8	77.5%	27.7%
12	1,000	1,560	8	3,120	8	79.7%	33.8%
12	1,500	1,560	8	3,120	8	80.7%	41.2%
12	2,000	1,560	8	3,120	8	81.5%	46.7%
12	2,500	1,560	8	3,120	8	82.0%	52.2%
12	3,500	1,560	8	3,120	8	82.2%	61.5%
12	4,000	1,560	8	3,120	8	82.2%	65.1%
12	4,000	1,560	8	3,120	8	82.2%	65.1%
12	4,500	1,560	8	3,120	8	82.2%	67.3%
12	5,000	1,560	8	3,120	8	82.1%	68.5%
12	5,500	1,560	8	3,120	8	82.1%	70.5%
12	6,000	1,560	8	3,120	8	82.1%	71.6%
12	6,500	1,560	8	3,120	8	81.9%	73.0%
12	7,000	1,560	8	3,120	8	81.9%	74.1%

Option Costs

						Northeast Pump Station	G370 Pump Station	G372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,560	8	3,120	8	$ 30,267,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 50,623,000
12	500	1,560	8	3,120	8	$ 37,250,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 57,606,000
12	1,000	1,560	8	3,120	8	$ 44,500,000	$ -	$ -	$ 13,856,000	$ 6,500,000	$ 64,856,000
12	1,500	1,560	8	3,120	8	$ 50,832,000	$ -	$ -	$ 13,856,000	$ 9,598,000	$ 74,286,000
12	2,000	1,560	8	3,120	8	$ 56,000,000	$ -	$ -	$ 13,856,000	$ 12,540,000	$ 82,396,000
12	2,500	1,560	8	3,120	8	$ 60,750,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 96,166,000
12	3,000	1,560	8	3,120	8	$ 64,722,000	$ -	$ -	$ 13,856,000	$ 21,560,000	$ 100,138,000
12	3,500	1,560	8	3,120	8	$ 80,000,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 124,471,000
12	3,600	1,560	8	3,120	8	$ 84,496,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 128,967,000
12	4,000	1,560	8	3,120	8	$ 87,500,000	$ -	$ -	$ 13,856,000	$ 30,615,000	$ 131,971,000
12	4,500	1,560	8	3,120	8	$ 91,000,000	$ -	$ -	$ 13,856,000	$ 36,719,000	$ 141,575,000
12	5,000	1,560	8	3,120	8	$ 94,647,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 153,003,000
12	5,500	1,560	8	3,120	8	$ 98,000,000	$ -	$ -	$ 13,856,000	$ 44,500,000	$ 156,356,000

Figure 6.5-25 Optimization for Priority Removals – Alternative A1-2B

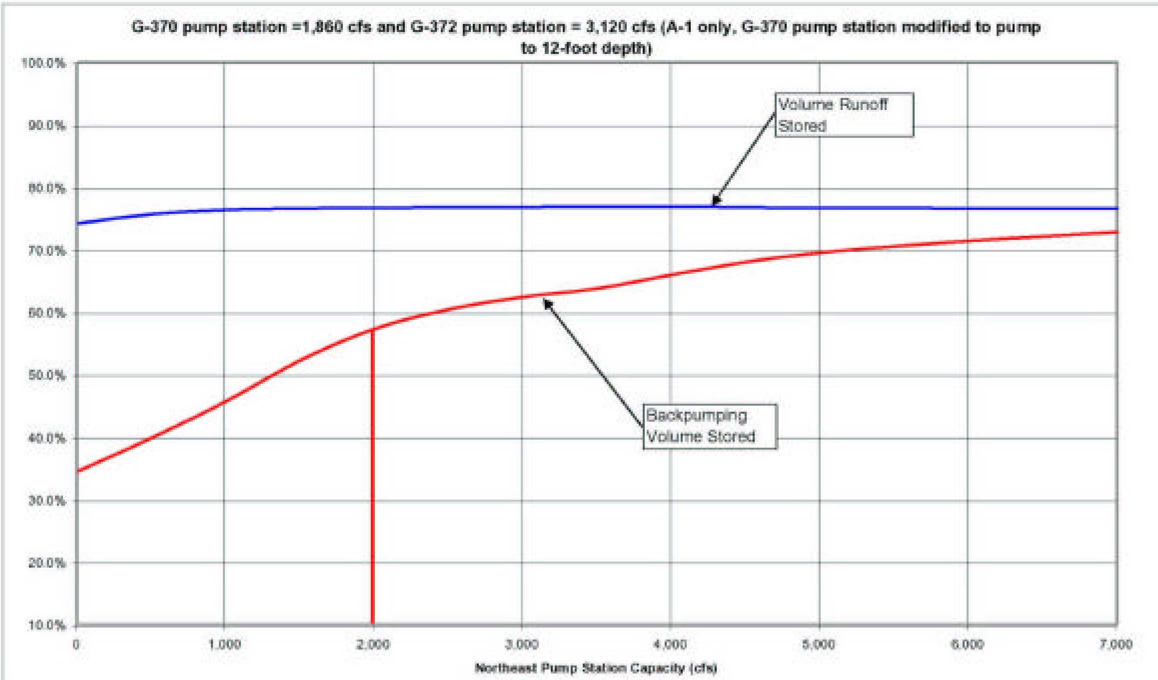


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	1,020	12	3,120	8	70.3%	20.9%
12	500	1,020	12	3,120	8	72.9%	28.9%
12	1,000	1,020	12	3,120	8	74.8%	36.2%
12	1,500	1,020	12	3,120	8	76.0%	41.6%
12	2,000	1,020	12	3,120	8	76.5%	48.4%
12	2,500	1,020	12	3,120	8	76.6%	54.2%
12	3,000	1,020	12	3,120	8	76.7%	58.8%
12	3,500	1,020	12	3,120	8	76.8%	61.4%
12	4,000	1,020	12	3,120	8	76.8%	63.1%
12	4,500	1,020	12	3,120	8	76.9%	64.3%
12	5,000	1,020	12	3,120	8	76.8%	66.6%
12	5,500	1,020	12	3,120	8	76.7%	68.5%
12	6,000	1,020	12	3,120	8	76.6%	69.8%
12	6,500	1,020	12	3,120	8	76.6%	70.8%
12	7,000	1,020	12	3,120	8	76.6%	71.6%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,020	12	3,120	8	$ 30,267,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 82,443,000
12	500	1,020	12	3,120	8	$ 37,250,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 89,426,000
12	1,000	1,020	12	3,120	8	$ 44,500,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 96,676,000
12	1,500	1,020	12	3,120	8	$ 50,832,000	$ 3,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 103,008,000
12	2,000	1,020	12	3,120	8	$ 56,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 9,598,000	$ 111,274,000
12	2,500	1,020	12	3,120	8	$ 60,750,000	$ 3,600,000	$ -	$ 42,076,000	$ 12,540,000	$ 118,966,000
12	3,000	1,020	12	3,120	8	$ 64,722,000	$ 3,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 131,958,000
12	3,500	1,020	12	3,120	8	$ 80,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 147,236,000
12	4,000	1,020	12	3,120	8	$ 87,500,000	$ 3,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 163,791,000
12	4,500	1,020	12	3,120	8	$ 91,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 167,291,000
12	5,000	1,020	12	3,120	8	$ 94,647,000	$ 3,600,000	$ -	$ 42,076,000	$ 36,719,000	$ 177,042,000
12	5,500	1,020	12	3,120	8	$ 98,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 188,176,000
12	6,000	1,020	12	3,120	8	$ 101,750,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 191,926,000
12	6,500	1,020	12	3,120	8	$ 105,000,000	$ 3,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 195,176,000

Figure 6.5-26 Optimization for Priority Removals – Alternative A1-2C

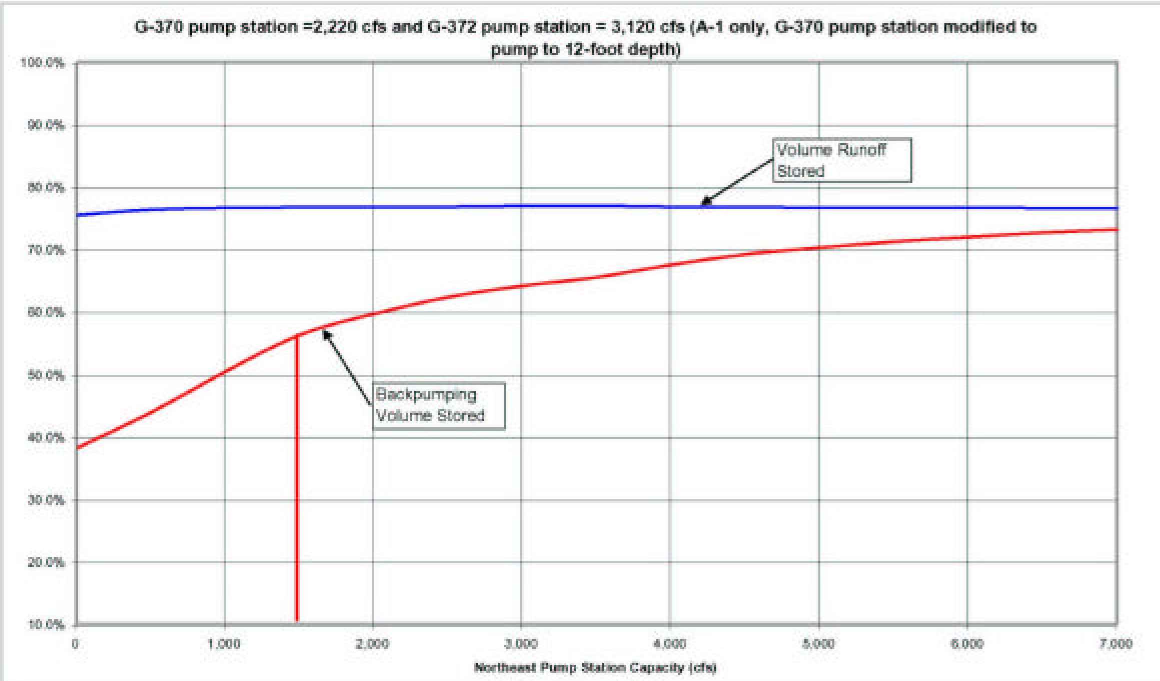


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	1,860	12	3,120	8	74.1%	34.5%
12	500	1,860	12	3,120	8	75.6%	39.8%
12	1,000	1,860	12	3,120	8	76.3%	45.7%
12	1,500	1,860	12	3,120	8	76.6%	52.2%
12	2,000	1,860	12	3,120	8	76.7%	57.3%
12	2,500	1,860	12	3,120	8	76.7%	60.4%
12	3,000	1,860	12	3,120	8	76.8%	62.4%
12	3,500	1,860	12	3,120	8	76.9%	63.7%
12	4,000	1,860	12	3,120	8	76.9%	65.9%
12	4,500	1,860	12	3,120	8	76.7%	68.0%
12	5,000	1,860	12	3,120	8	76.7%	69.4%
12	5,500	1,860	12	3,120	8	76.6%	70.5%
12	6,000	1,860	12	3,120	8	76.6%	71.4%
12	6,500	1,860	12	3,120	8	76.6%	72.1%
12	7,000	1,860	12	3,120	8	76.5%	72.8%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,860	12	3,120	8	$ 30,267,000	$ 10,200,000	$ -	$ 42,076,000	$ 6,500,000	$ 89,043,000
12	500	1,860	12	3,120	8	$ 37,250,000	$ 10,200,000	$ -	$ 42,076,000	$ 6,500,000	$ 96,026,000
12	1,000	1,860	12	3,120	8	$ 44,500,000	$ 10,200,000	$ -	$ 42,076,000	$ 9,598,000	$ 106,374,000
12	1,500	1,860	12	3,120	8	$ 50,832,000	$ 10,200,000	$ -	$ 42,076,000	$ 9,598,000	$ 112,706,000
12	2,000	1,860	12	3,120	8	$ 56,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 129,836,000
12	2,500	1,860	12	3,120	8	$ 60,750,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 134,586,000
12	3,000	1,860	12	3,120	8	$ 64,722,000	$ 10,200,000	$ -	$ 42,076,000	$ 21,560,000	$ 138,558,000
12	3,500	1,860	12	3,120	8	$ 80,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 30,615,000	$ 162,891,000
12	4,000	1,860	12	3,120	8	$ 87,500,000	$ 10,200,000	$ -	$ 42,076,000	$ 36,719,000	$ 176,495,000
12	4,500	1,860	12	3,120	8	$ 91,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 36,719,000	$ 179,995,000
12	5,000	1,860	12	3,120	8	$ 94,647,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 191,423,000
12	5,500	1,860	12	3,120	8	$ 98,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 194,776,000
12	6,000	1,860	12	3,120	8	$ 101,750,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 198,526,000
12	6,500	1,860	12	3,120	8	$ 105,000,000	$ 10,200,000	$ -	$ 42,076,000	$ 44,500,000	$ 201,776,000

Figure 6.5-27 Optimization for Priority Removals – Alternative A1-2D

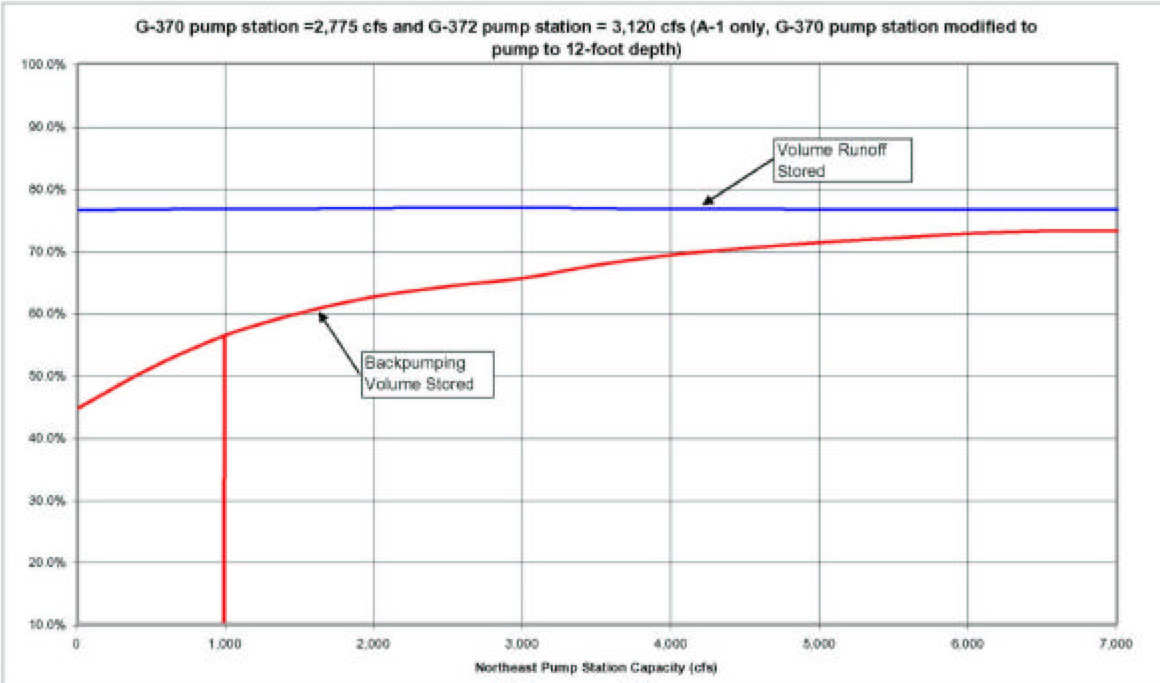


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	2,220	12	3,120	8	75.4%	38.1%
12	500	2,220	12	3,120	8	76.3%	43.8%
12	1,000	2,220	12	3,120	8	76.8%	50.3%
12	1,500	2,220	12	3,120	8	76.7%	56.2%
12	2,000	2,220	12	3,120	8	76.7%	59.6%
12	2,500	2,220	12	3,120	8	76.7%	62.3%
12	3,000	2,220	12	3,120	8	76.8%	64.0%
12	3,500	2,220	12	3,120	8	76.9%	65.4%
12	4,000	2,220	12	3,120	8	76.7%	67.4%
12	4,500	2,220	12	3,120	8	76.7%	69.1%
12	5,000	2,220	12	3,120	8	76.6%	70.2%
12	5,500	2,220	12	3,120	8	76.6%	71.1%
12	6,000	2,220	12	3,120	8	76.6%	71.9%
12	6,500	2,220	12	3,120	8	76.5%	72.6%
12	7,000	2,220	12	3,120	8	76.5%	73.1%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,220	12	3,120	8	$ 30,267,000	$ 11,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 90,443,000
12	500	2,220	12	3,120	8	$ 37,250,000	$ 11,600,000	$ -	$ 42,076,000	$ 6,500,000	$ 97,426,000
12	1,000	2,220	12	3,120	8	$ 44,500,000	$ 11,600,000	$ -	$ 42,076,000	$ 9,598,000	$ 107,774,000
12	1,500	2,220	12	3,120	8	$ 50,832,000	$ 11,600,000	$ -	$ 42,076,000	$ 12,540,000	$ 117,048,000
12	2,000	2,220	12	3,120	8	$ 56,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 131,236,000
12	2,500	2,220	12	3,120	8	$ 60,750,000	$ 11,600,000	$ -	$ 42,076,000	$ 21,560,000	$ 135,986,000
12	3,000	2,220	12	3,120	8	$ 64,722,000	$ 11,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 149,013,000
12	3,500	2,220	12	3,120	8	$ 80,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 30,615,000	$ 164,291,000
12	4,000	2,220	12	3,120	8	$ 87,500,000	$ 11,600,000	$ -	$ 42,076,000	$ 36,719,000	$ 177,895,000
12	4,500	2,220	12	3,120	8	$ 91,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 189,176,000
12	5,000	2,220	12	3,120	8	$ 94,647,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 192,823,000
12	5,500	2,220	12	3,120	8	$ 98,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 196,176,000
12	6,000	2,220	12	3,120	8	$ 101,750,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 199,926,000
12	6,500	2,220	12	3,120	8	$ 105,000,000	$ 11,600,000	$ -	$ 42,076,000	$ 44,500,000	$ 203,176,000

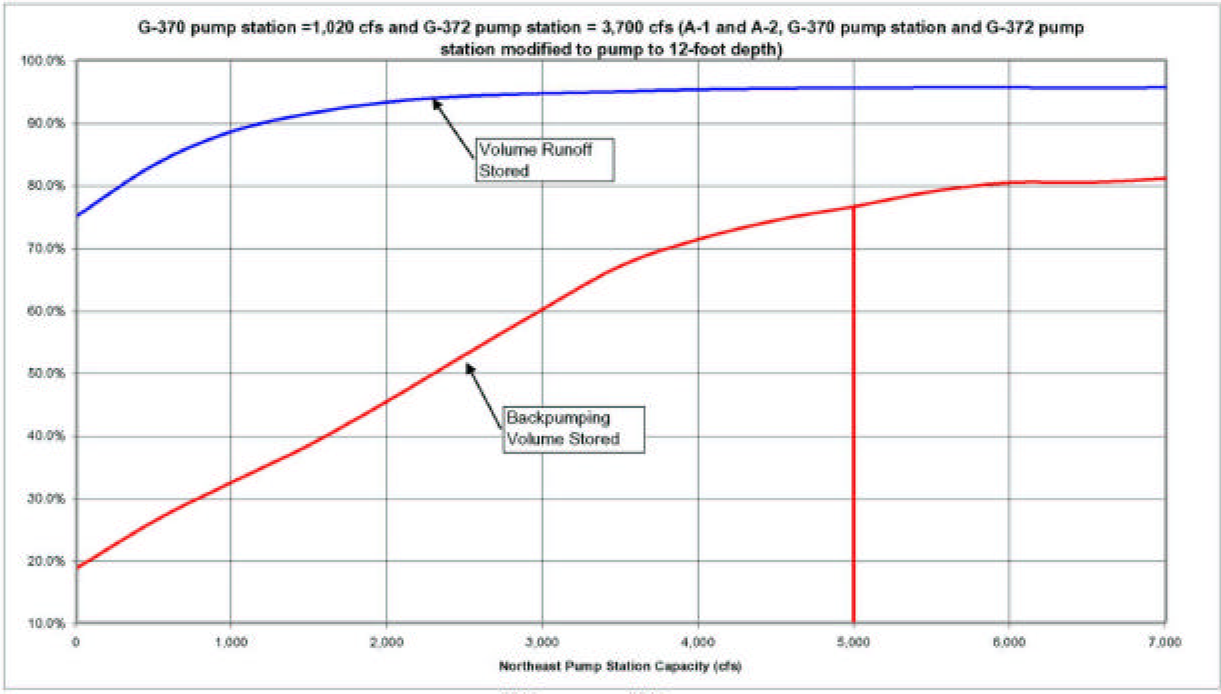
Figure 6.5-28 Optimization for Priority Removals – Alternative A1-2E



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	2,775	12	3,120	8	76.4%	44.6%
12	500	2,775	12	3,120	8	76.6%	51.2%
12	1,000	2,775	12	3,120	8	76.7%	56.4%
12	1,500	2,775	12	3,120	8	76.7%	59.9%
12	2,000	2,775	12	3,120	8	76.7%	62.6%
12	2,500	2,775	12	3,120	8	76.9%	64.2%
12	3,000	2,775	12	3,120	8	76.9%	65.5%
12	3,500	2,775	12	3,120	8	76.7%	67.6%
12	4,000	2,775	12	3,120	8	76.7%	69.2%
12	4,500	2,775	12	3,120	8	76.6%	70.3%
12	5,000	2,775	12	3,120	8	76.6%	71.2%
12	5,500	2,775	12	3,120	8	76.6%	72.0%
12	6,000	2,775	12	3,120	8	76.5%	72.7%
12	6,500	2,775	12	3,120	8	76.5%	73.1%
12	7,000	2,775	12	3,120	8	76.5%	73.1%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,775	12	3,120	8	$ 30,267,000	$ 13,100,000	$ -	$ 42,076,000	$ 6,500,000	$ 91,943,000
12	500	2,775	12	3,120	8	$ 37,250,000	$ 13,100,000	$ -	$ 42,076,000	$ 9,598,000	$ 102,024,000
12	1,000	2,775	12	3,120	8	$ 44,500,000	$ 13,100,000	$ -	$ 42,076,000	$ 12,540,000	$ 112,216,000
12	1,500	2,775	12	3,120	8	$ 50,832,000	$ 13,100,000	$ -	$ 42,076,000	$ 21,560,000	$ 127,568,000
12	2,000	2,775	12	3,120	8	$ 56,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 21,560,000	$ 132,736,000
12	2,500	2,775	12	3,120	8	$ 60,750,000	$ 13,100,000	$ -	$ 42,076,000	$ 30,615,000	$ 146,541,000
12	3,000	2,775	12	3,120	8	$ 64,722,000	$ 13,100,000	$ -	$ 42,076,000	$ 30,615,000	$ 150,513,000
12	3,500	2,775	12	3,120	8	$ 80,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 36,719,000	$ 171,895,000
12	4,000	2,775	12	3,120	8	$ 87,500,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 187,176,000
12	4,500	2,775	12	3,120	8	$ 91,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 190,676,000
12	5,000	2,775	12	3,120	8	$ 94,647,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 194,323,000
12	5,500	2,775	12	3,120	8	$ 98,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 197,676,000
12	6,000	2,775	12	3,120	8	$ 101,750,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 201,426,000
12	6,500	2,775	12	3,120	8	$ 105,000,000	$ 13,100,000	$ -	$ 42,076,000	$ 44,500,000	$ 204,676,000

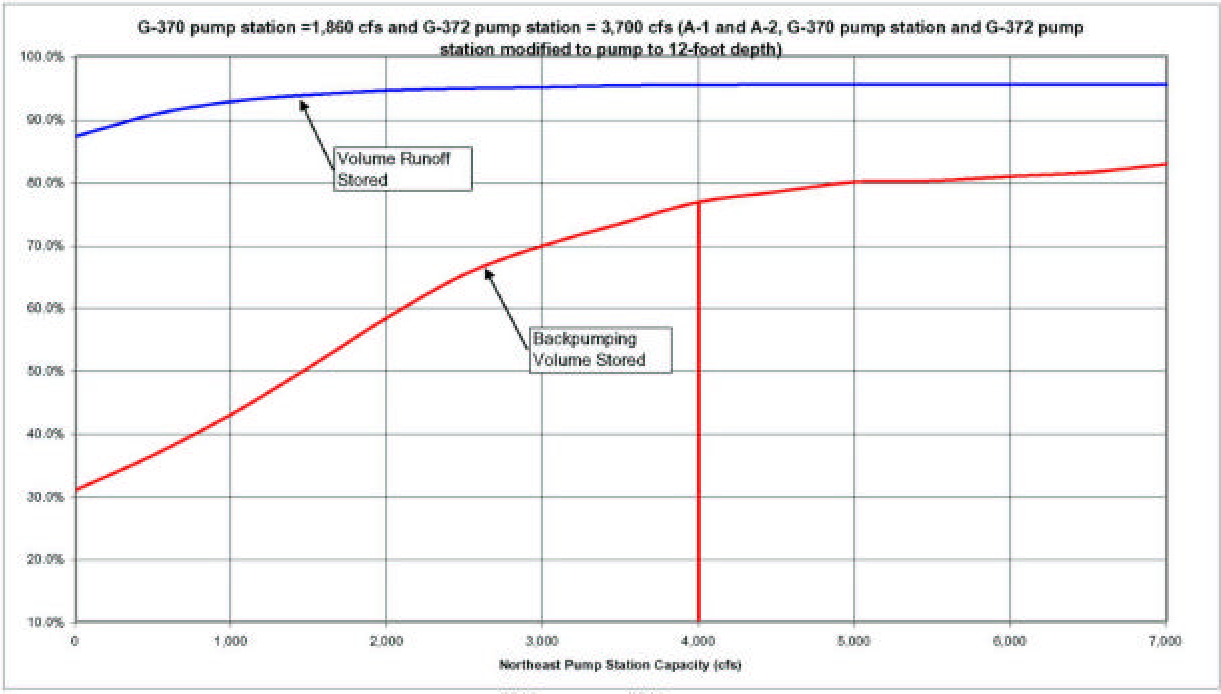
Figure 6.5-29 Optimization for Priority Removals – Alternative A2-2B



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	1,020	12	3,700	12	75.0%	18.7%
12	500	1,020	12	3,700	12	83.2%	26.2%
12	1,000	1,020	12	3,700	12	88.5%	32.5%
12	1,500	1,020	12	3,700	12	91.4%	38.4%
12	2,000	1,020	12	3,700	12	93.2%	45.4%
12	2,500	1,020	12	3,700	12	94.1%	52.9%
12	3,000	1,020	12	3,700	12	94.5%	60.2%
12	3,500	1,020	12	3,700	12	94.8%	67.0%
12	4,000	1,020	12	3,700	12	95.2%	71.3%
12	4,500	1,020	12	3,700	12	95.3%	74.3%
12	5,000	1,020	12	3,700	12	95.4%	76.5%
12	5,500	1,020	12	3,700	12	95.5%	78.9%
12	6,000	1,020	12	3,700	12	95.5%	80.3%
12	6,500	1,020	12	3,700	12	95.5%	80.3%
12	7,000	1,020	12	3,700	12	95.5%	80.9%

Option Costs						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,020	12	3,700	12	$ 30,267,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 144,155,000
12	500	1,020	12	3,700	12	$ 37,250,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 151,138,000
12	1,000	1,020	12	3,700	12	$ 44,500,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 158,388,000
12	1,500	1,020	12	3,700	12	$ 50,832,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 164,720,000
12	2,000	1,020	12	3,700	12	$ 56,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 172,986,000
12	2,500	1,020	12	3,700	12	$ 60,750,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 180,678,000
12	3,000	1,020	12	3,700	12	$ 64,722,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 193,670,000
12	3,500	1,020	12	3,700	12	$ 80,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 208,948,000
12	4,000	1,020	12	3,700	12	$ 87,500,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 225,503,000
12	4,500	1,020	12	3,700	12	$ 91,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 229,003,000
12	5,000	1,020	12	3,700	12	$ 94,647,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 238,754,000
12	5,500	1,020	12	3,700	12	$ 98,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 249,888,000
12	6,000	1,020	12	3,700	12	$ 101,750,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 253,638,000
12	6,500	1,020	12	3,700	12	$ 105,000,000	$ 10,200,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 256,888,000

Figure 6.5-30 Optimization for Priority Removals – Alternative A2-2C

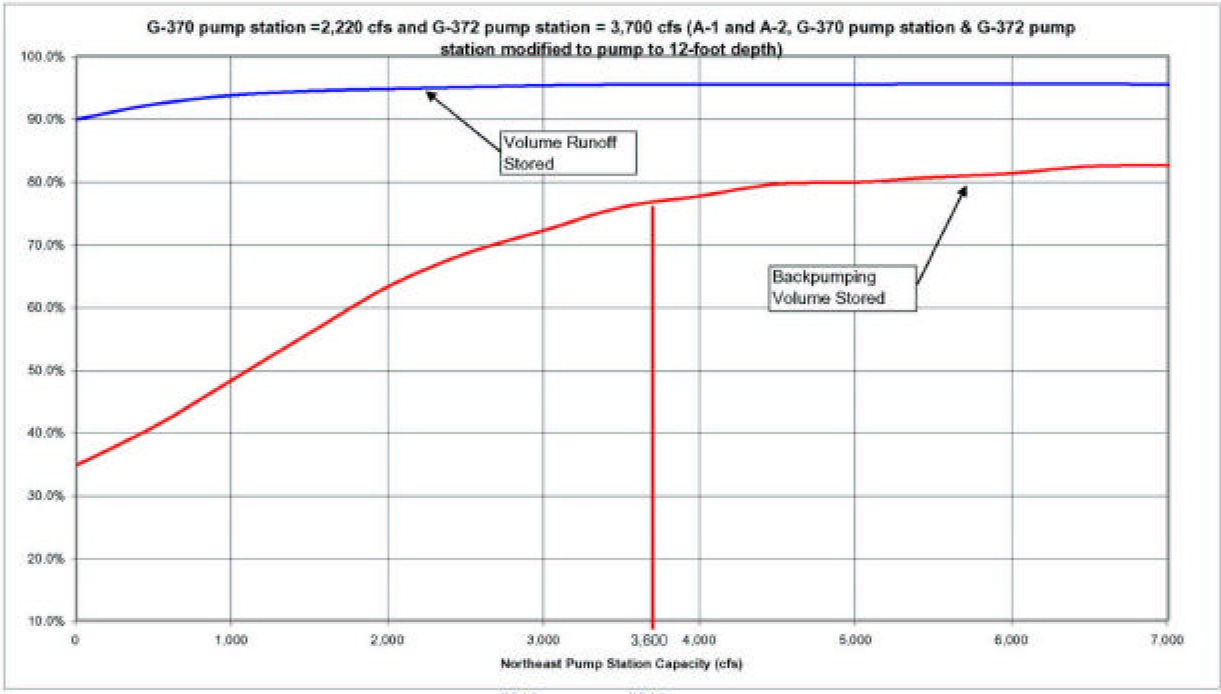


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	1,860	12	3,700	12	87.1%	30.9%
12	500	1,860	12	3,700	12	90.6%	36.5%
12	1,000	1,860	12	3,700	12	92.7%	42.9%
12	1,500	1,860	12	3,700	12	93.7%	50.5%
12	2,000	1,860	12	3,700	12	94.4%	58.4%
12	2,500	1,860	12	3,700	12	94.7%	65.3%
12	3,000	1,860	12	3,700	12	94.9%	69.8%
12	3,500	1,860	12	3,700	12	95.2%	73.3%
12	4,000	1,860	12	3,700	12	95.3%	76.7%
12	4,500	1,860	12	3,700	12	95.4%	78.3%
12	5,000	1,860	12	3,700	12	95.4%	79.8%
12	5,500	1,860	12	3,700	12	95.4%	80.0%
12	6,000	1,860	12	3,700	12	95.4%	80.8%
12	6,500	1,860	12	3,700	12	95.4%	81.4%
12	7,000	1,860	12	3,700	12	95.4%	82.7%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	1,860	12	3,700	12	$ 30,267,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 145,555,000
12	500	1,860	12	3,700	12	$ 37,250,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 152,538,000
12	1,000	1,860	12	3,700	12	$ 44,500,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 162,886,000
12	1,500	1,860	12	3,700	12	$ 50,832,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 169,218,000
12	2,000	1,860	12	3,700	12	$ 56,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 186,348,000
12	2,500	1,860	12	3,700	12	$ 60,750,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 191,098,000
12	3,000	1,860	12	3,700	12	$ 64,722,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 195,070,000
12	3,500	1,860	12	3,700	12	$ 80,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 219,403,000
12	4,000	1,860	12	3,700	12	$ 87,500,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 233,007,000
12	4,500	1,860	12	3,700	12	$ 91,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 236,507,000
12	5,000	1,860	12	3,700	12	$ 94,647,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 247,935,000
12	5,500	1,860	12	3,700	12	$ 98,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 251,288,000
12	6,000	1,860	12	3,700	12	$ 101,750,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 255,038,000
12	6,500	1,860	12	3,700	12	$ 105,000,000	$ 11,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 258,288,000

Figure 6.5-31 Optimization for Priority Removals – Alternative A2-2D

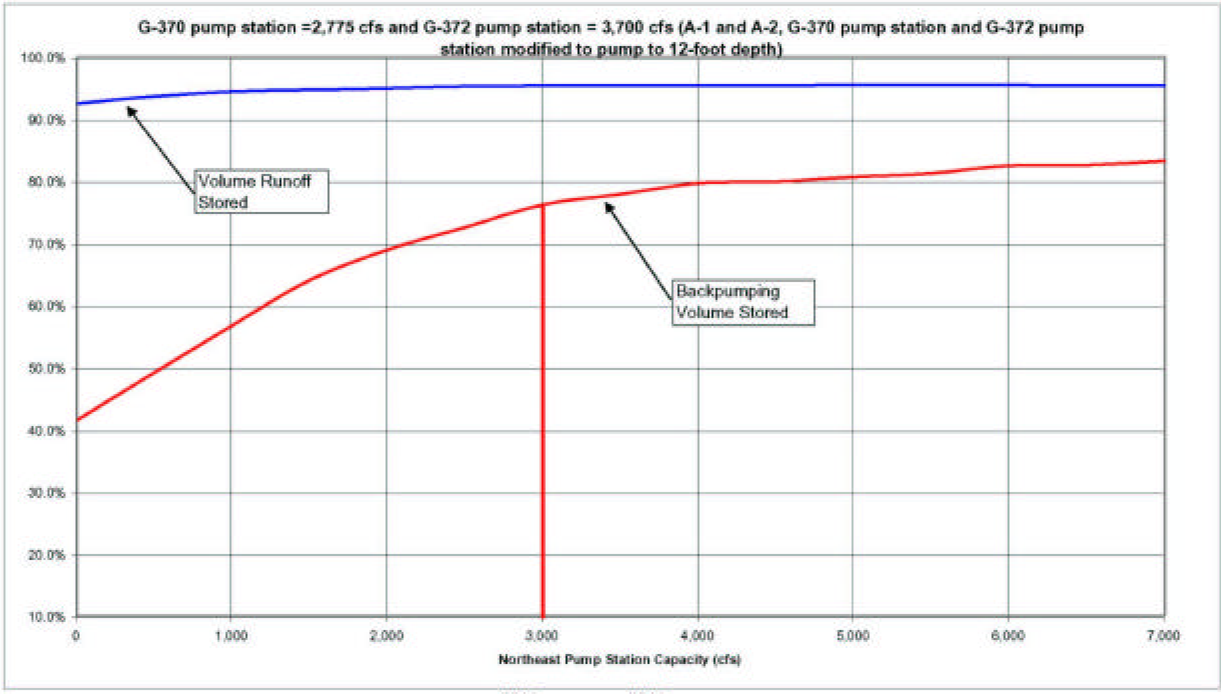


Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	2,220	12	3,700	12	89.8%	34.6%
12	500	2,220	12	3,700	12	92.2%	40.9%
12	1,000	2,220	12	3,700	12	93.6%	48.3%
12	1,500	2,220	12	3,700	12	94.3%	55.8%
12	2,000	2,220	12	3,700	12	94.6%	63.2%
12	2,500	2,220	12	3,700	12	94.9%	68.4%
12	3,000	2,220	12	3,700	12	95.2%	72.1%
12	3,500	2,220	12	3,700	12	95.3%	75.8%
12	4,000	2,220	12	3,700	12	95.4%	77.6%
12	4,500	2,220	12	3,700	12	95.4%	79.5%
12	5,000	2,220	12	3,700	12	95.4%	79.8%
12	5,500	2,220	12	3,700	12	95.4%	80.6%
12	6,000	2,220	12	3,700	12	95.4%	81.2%
12	6,500	2,220	12	3,700	12	95.4%	82.3%
12	7,000	2,220	12	3,700	12	95.4%	82.4%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,220	12	3,700	12	$ 30,267,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 147,055,000
12	500	2,220	12	3,700	12	$ 37,250,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 154,038,000
12	1,000	2,220	12	3,700	12	$ 44,500,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 164,386,000
12	1,500	2,220	12	3,700	12	$ 50,832,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 173,660,000
12	2,000	2,220	12	3,700	12	$ 56,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 187,848,000
12	2,500	2,220	12	3,700	12	$ 60,750,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 192,598,000
12	3,000	2,220	12	3,700	12	$ 64,722,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 205,625,000
12	3,500	2,220	12	3,700	12	$ 80,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 220,903,000
12	4,000	2,220	12	3,700	12	$ 87,500,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 234,507,000
12	4,500	2,220	12	3,700	12	$ 91,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 245,788,000
12	5,000	2,220	12	3,700	12	$ 94,647,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 249,435,000
12	5,500	2,220	12	3,700	12	$ 98,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 252,788,000
12	6,000	2,220	12	3,700	12	$ 101,750,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 256,538,000
12	6,500	2,220	12	3,700	12	$ 105,000,000	$ 13,100,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 259,788,000

Figure 6.5-32 Optimization for Priority Removals – Alternative A2-2E



Water Depth	Northeast Pump Station (cfs)	G-370 (cfs)	G-370 Pump Head	G-372 (cfs)	G-372 Pump Head	2015 Percentage of Runoff Stored	2015 Backpumping Volume Stored
12	0	2,775	12	3,700	12	92.4%	41.5%
12	500	2,775	12	3,700	12	93.6%	49.2%
12	1,000	2,775	12	3,700	12	94.4%	56.8%
12	1,500	2,775	12	3,700	12	94.6%	64.1%
12	2,000	2,775	12	3,700	12	94.9%	68.9%
12	2,500	2,775	12	3,700	12	95.2%	72.6%
12	3,000	2,775	12	3,700	12	95.3%	76.2%
12	3,500	2,775	12	3,700	12	95.4%	77.8%
12	4,000	2,775	12	3,700	12	95.4%	79.6%
12	4,500	2,775	12	3,700	12	95.4%	79.9%
12	5,000	2,775	12	3,700	12	95.4%	80.6%
12	5,500	2,775	12	3,700	12	95.4%	81.2%
12	6,000	2,775	12	3,700	12	95.4%	82.4%
12	6,500	2,775	12	3,700	12	95.4%	82.5%
12	7,000	2,775	12	3,700	12	95.4%	83.2%

Option Costs

						Northeast Pump Station	G-370 Pump Station	G-372 Pump Station	Gate Structures	Canal Modifications	Total Cost
12	0	2,775	12	3,700	12	$ 30,267,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 6,500,000	$ 137,555,000
12	500	2,775	12	3,700	12	$ 37,250,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 9,598,000	$ 147,636,000
12	1,000	2,775	12	3,700	12	$ 44,500,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 12,540,000	$ 157,828,000
12	1,500	2,775	12	3,700	12	$ 50,832,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 173,180,000
12	2,000	2,775	12	3,700	12	$ 56,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 21,560,000	$ 178,348,000
12	2,500	2,775	12	3,700	12	$ 60,750,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 192,153,000
12	3,000	2,775	12	3,700	12	$ 64,722,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 30,615,000	$ 196,125,000
12	3,500	2,775	12	3,700	12	$ 80,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 36,719,000	$ 217,507,000
12	4,000	2,775	12	3,700	12	$ 87,500,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 232,788,000
12	4,500	2,775	12	3,700	12	$ 91,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 236,288,000
12	5,000	2,775	12	3,700	12	$ 94,647,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 239,935,000
12	5,500	2,775	12	3,700	12	$ 98,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 243,288,000
12	6,000	2,775	12	3,700	12	$ 101,750,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 247,038,000
12	6,500	2,775	12	3,700	12	$ 105,000,000	$ 3,600,000	$ 17,000,000	$ 80,188,000	$ 44,500,000	$ 250,288,000

6.5.3 Recommended Alternative

The optimization goal of installing the most cost effective pump capacity to meet maximum deliveries is contradictory with the goal of maximizing pump capacity to capture local runoff, pump backs, and Lake Okeechobee regulatory releases:

- Based on the application of the WBM over the POS, a northeast pump station sized for 1,500 cfs working with G-370 and G-372 pump stations unmodified would provide sufficient capacity during the first phase of operation to provide the maximum delivery percentages that can be expected with an EAA Reservoir A-1 of 190,000 acre feet of storage volume. Further modifications to the NNRC and to G-370 and G-372 pump stations to allow pumping capacities of 2,220 and 3,700 cfs respectively to full EAA Reservoir A-1 water depth would provide the additional capacity needed for the second phase of operation.
- Based on the application of the WBM over the POS, a northeast pump station sized for 3,500 to 4,000 cfs working with G-370 and G-372 pump stations unmodified would be required to provide sufficient capacity during the first phase to maximize local runoff and pump back capture. NNRC modifications would also be required to increase conveyance capacity. Further modifications to G-370 and G-372 pump stations to allow pumping capacities to full EAA Reservoir A-1 water depth would provide the additional capacity needed for the second phase of operation.

The second alternative has several advantages over the first:

- A 3,500 to 4,000 cfs pump station sized to maximize capture of priority removals can meet all of the delivery goals that a 1,500 cfs station would meet. The converse is not true; the 1,500 cfs station could not provide the same priority removal levels.
- A 3,500 to 4,000 cfs pump station would provide a significant increase in flood protection capacity. Under phase one, if EAA Reservoir A-1 water levels are greater than 8 feet, G-370 pump station would be limited to pumping to STA-3/4. If, for operational reasons, it is preferable not to discharge water to the STA-3/4, the northeast pump station would provide substantial capacity for flood protection, greater than is now available. Under the same circumstances, a 1,500 cfs pump station could not provide a similar level of flood protection.
- The S-2 pump station which currently provides pump backs to Lake Okeechobee has a capacity of 3,600 cfs. Intuitively, a comparably sized pump station would be needed to minimize pump backs.
- Modification of G-370 pump station for the second phase will require that the pumps in that station be removed from service for periods of time. In addition, the infrastructure required to allow isolation of the Supply Canal may require periods when the entire pump station will need to be removed from service. Having a substantial pumping capability in the northeast pump station will ease the disruption that will be experienced during those modifications. A 1,500 cfs northeast pump station would limit flood protection during construction of future G-370 pump station modifications.

- Figure 6.5-32 shows the percent of time for which flow is greater than a given amount during the period of simulation used for the WBM. For 2010 flow conditions, available flow rates are greater than the combined capacity (3,065 cfs) of a 1,500 cfs northeast pump station and G-370 pump station 1.6 percent of the time as compared to only 0.2 percent for the larger pump station. Figure 6.5-33 shows the same for 2015 flow conditions. The available flow rates greater than the combined capacity (3,720 cfs) of a 1,500 cfs northeast pump station and G-370 pump station increases to 4.3 percent, while that for the 3,500 to 4,000 cfs station increases only to 0.43 percent. This indicates that a 3,500 to 4,000 cfs pump station would provide the capacity needed to capture most flows experienced.
- The period of simulation used for the WBM provides daily amounts and does not account for peak flows within a storm event. While some peak flow rates would not be captured with a 3,500 to 4,000 cfs pump station, a relatively large number of peak flows would not be captured with a 1,500 cfs pump station.

Figure 6.5-33 NNRC Flow vs. Percentage Greater from POS (2010)

Flows >=	POS % NNRC	NNRC No. Days
0	100.00%	13149
500	32.83%	4317
1000	25.81%	3394
1200	20.78%	2732
1500	7.35%	967
1800	5.98%	786
2000	5.10%	671
2200	4.46%	587
2500	3.58%	471
2800	1.74%	229
3000	1.61%	212
3200	1.38%	182
3500	1.23%	162
3800	1.03%	135
4000	0.92%	121
4200	0.78%	103
4500	0.65%	85
4800	0.58%	76
5000	0.53%	70
5200	0.50%	66
5500	0.41%	54
5800	0.24%	32
6000	0.20%	26
6200	0.17%	22
6500	0.13%	17
6800	0.11%	14
7000	0.08%	11
7200	0.08%	11
7500	0.08%	10
7800	0.06%	8
8000	0.06%	8
8200	0.06%	8
8500	0.06%	8
8800	0.05%	7
9000	0.05%	6
9150	0.02%	3
9151	0.00%	0

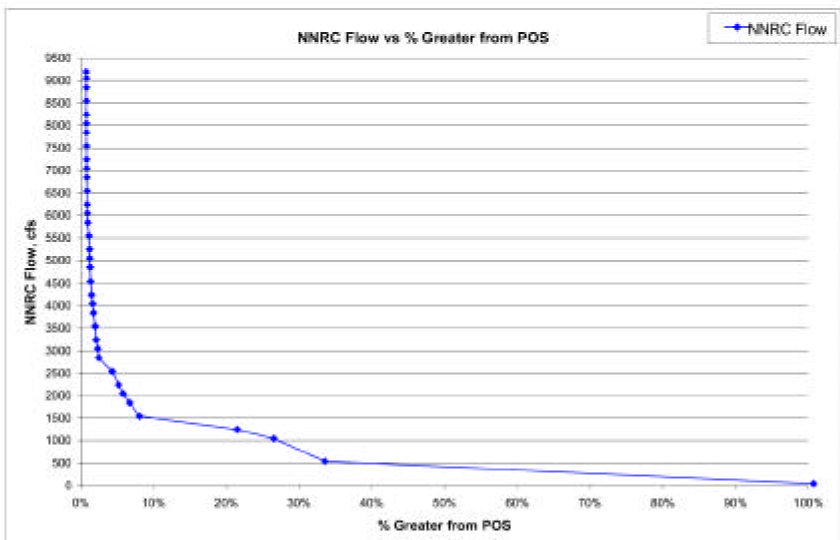
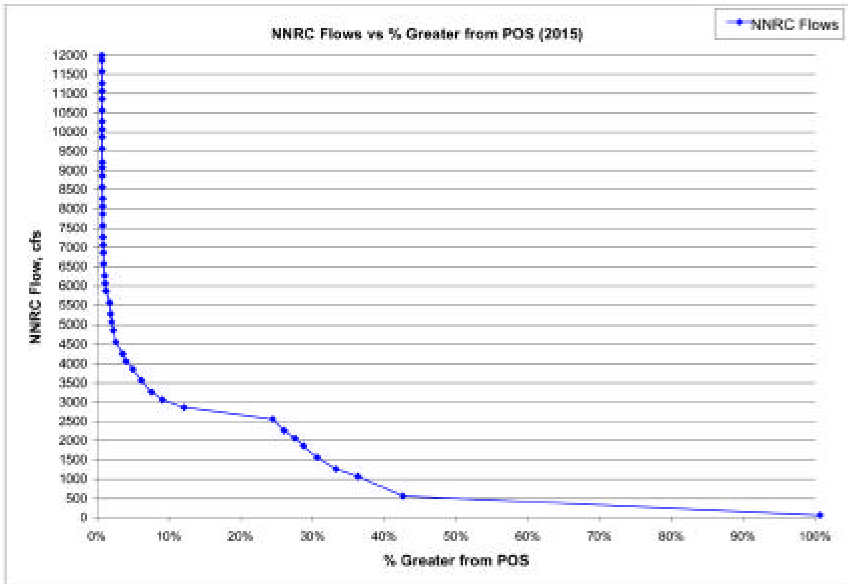


Figure 6.5-34 NNRC Flows vs. Percentage Greater from POS (2015)

2015

Flows >=	POS % NNRC	NNRC No. Days
0	100.00%	13149
500	41.89%	5508
1000	35.65%	4687
1200	32.55%	4280
1500	29.91%	3933
1800	28.02%	3685
2000	26.87%	3533
2200	25.34%	3332
2500	23.74%	3121
2800	11.45%	1505
3000	8.42%	1107
3200	6.87%	903
3500	5.46%	718
3800	4.30%	565
4000	3.30%	434
4200	2.87%	378
4500	1.91%	251
4800	1.57%	207
5000	1.34%	176
5200	1.20%	158
5500	1.03%	136
5800	0.56%	73
6000	0.43%	57
6200	0.38%	50
6500	0.24%	31
6800	0.19%	25
7000	0.17%	22
7200	0.15%	20
7500	0.12%	16
7800	0.11%	15
8000	0.11%	14
8200	0.11%	14
8500	0.07%	9
8800	0.06%	8
9000	0.05%	7
9150	0.05%	7
9500	0.04%	5
9800	0.03%	4
10000	0.03%	4
10200	0.02%	3
10500	0.02%	2
10800	0.02%	2
11000	0.02%	2
11200	0.02%	2
11500	0.01%	1
11800	0.01%	1
11925	0.01%	1
11926	0.00%	0



The primary disadvantage for the second alternative is cost. In addition to the costs associated with a smaller pump station, the 1,500 cfs pump station can provide the optimum deliveries without canal modification for the first phase, and minimal canal modification for the second phase. To be effective, the larger pump station would require canal modifications that coincide with the first phase of construction.

We recommend:

- Construction of a 3,600 cfs northeast pump station concurrent to the construction of EAA Reservoir A-1.
- G-370 and G-372 pump stations be used unmodified during phase one operation to pump into the EAA Reservoir A-1 when EAA Reservoir A-1 water levels are less than eight feet and directly to the STA-3/4 Supply Canal when EAA Reservoir A-1 water levels are greater than eight feet.
- The G-370 and G-372 pump stations be modified to pump 2,220 and 3,700 cfs capacity to full EAA Reservoir A-1 depth as part of the second phase construction.
- Canal modifications to provide matching conveyance capacity during phase one.

6.6 GATES

Gate structures will be used to release water from the EAA Reservoir A-1 to the NNRC and STA-3/4 Supply Canal for environmental, agricultural, and emergency release purposes. In addition, gate structures along the STA-3/4 Supply Canal will be used to help fill the EAA

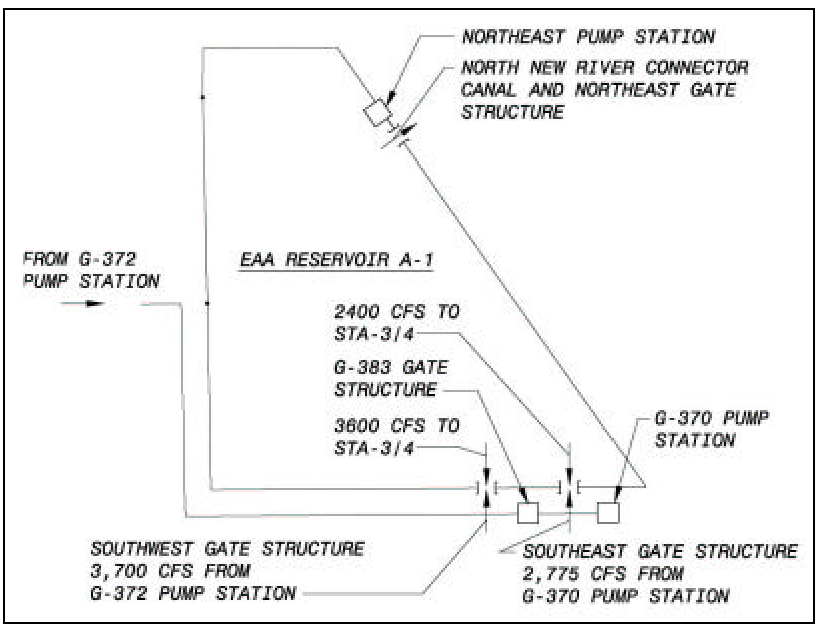
Reservoir A-1. The SFWMD is most familiar with vertical roller lift gates, and because of the need for bidirectional flow, these will be the gates used for the EAA Reservoir A-1. Additional information regarding the mechanical requirements of the EAA Reservoir A-1 gated culverts structures is discussed in Section 13.4.

Figure 6.6-1 shows a potential layout for gates at the EAA Reservoir A-1. It includes a gate structure located at the northeast corner of the EAA Reservoir A-1 to release flows to the NNRC for both agricultural and emergency release purposes. It shows southeast and southwest gate structures used to discharge flows for emergency release and environmental purposes to STA-3/4 while also serving as inflow gates from the STA-3/4 Supply Canal to the EAA Reservoir A-1. The requirements for each of these gate structures are detailed in the following sections.

The existing G-383 gate structure was installed to allow isolation of the eastern and western flow ways of STA-3/4. The structure has limited hydraulic capacity and would not allow the full capacity of either pump station to pass. It has been assumed that it will remain in service and will normally be closed, thereby requiring that two new gate structures be constructed in the north embankment of the STA-3/4 Supply Canal.

A backup power supply will be provided for each gate structure. The method for providing back up power will be standby generators or a separate power supply from the nearest pump station, depending upon the location of the gate structure.

Figure 6.6-1 Phase 1 EAA Reservoir A-1 Control Structures Location Map



6.6.1.1 Northeast Gate Structure

The northeast gate structure will be located close to the northeast pump station. It is required to meet the agricultural deliveries to the EAA served by the EAA Reservoir A-1. Agricultural deliveries from the EAA Reservoir A-1 to the EAA system ranges from 0 cfs in the wet season, to approximately 1,960 cfs during the dry season based on the period of simulation. A variety of gate sizes to meet this operational range were investigated; some of these results are presented in Figures 6.6-2 and 6.6-3. The gates shown in these figures are submerged gates (i.e. orifice flow);

with inverts located 10 feet below the bottom of the EAA Reservoir A-1, or approximately at -1.4 NAVD88. The gated culvert structures being considered for the EAA Reservoir A-1 are 10 feet by 10 feet culverts with roller gates on the external embankment. If a 50-foot gate is needed, then five parallel 10 feet by 10 feet gated culvert structures will be placed in that location. For the northeast gate structure, it was determined that a 50-foot wide bank of 10 feet by 10 feet gated culverts are needed in order meet agricultural deliveries with 0.5 feet of head loss across structures. The required water surface elevation in the EAA Reservoir A-1 depends on the necessary back water condition in the NNRC. For irrigators to have sufficient suction conditions for their irrigation pumps, the minimum water surface elevation in the NNRC is 10 to 10.5 NAVD88.

The scatter-points that are shown on Figures 6.6-2 and 6.6-3 correspond to agricultural deliveries predicted by the Water Balance Model for the conditions recommended: northeast pump station capacity of 3,600 cfs, G-370 pump station unmodified capacity of 2,340 cfs, and G-372 pump station unmodified capacity of 3,120 cfs. Points, which lie above a gate opening curve, occur when the EAA Reservoir A-1 stage is sufficient to overcome the head loss incurred through the gate and suction requirements of irrigation pumps to meet agricultural demands by gravity flow. Points which lie below the curves, occur when there is insufficient head differential available to meet an agricultural delivery and pumping is necessary to release flow to meet agricultural demands.

The northeast gate structure can also help to release excess floodwater from the EAA Reservoir A-1, however downstream canal conditions will dictate the maximum outflow from the northeast gate structure. By designing the gate to meet agricultural demands with 0.5 feet of head loss, the northeast gate structure will have adequate capacity to also meet potential emergency release demands.

Figure 6.6-2 Northeast Gate Submerged Gate Curve 50-Foot Wide Gate

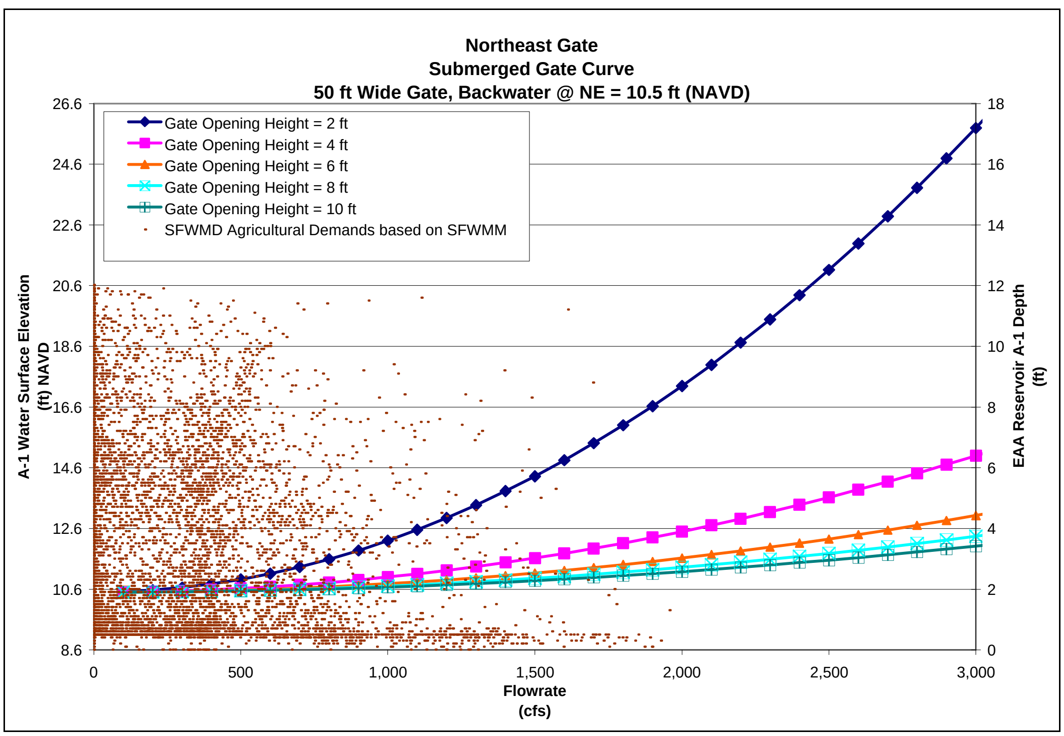
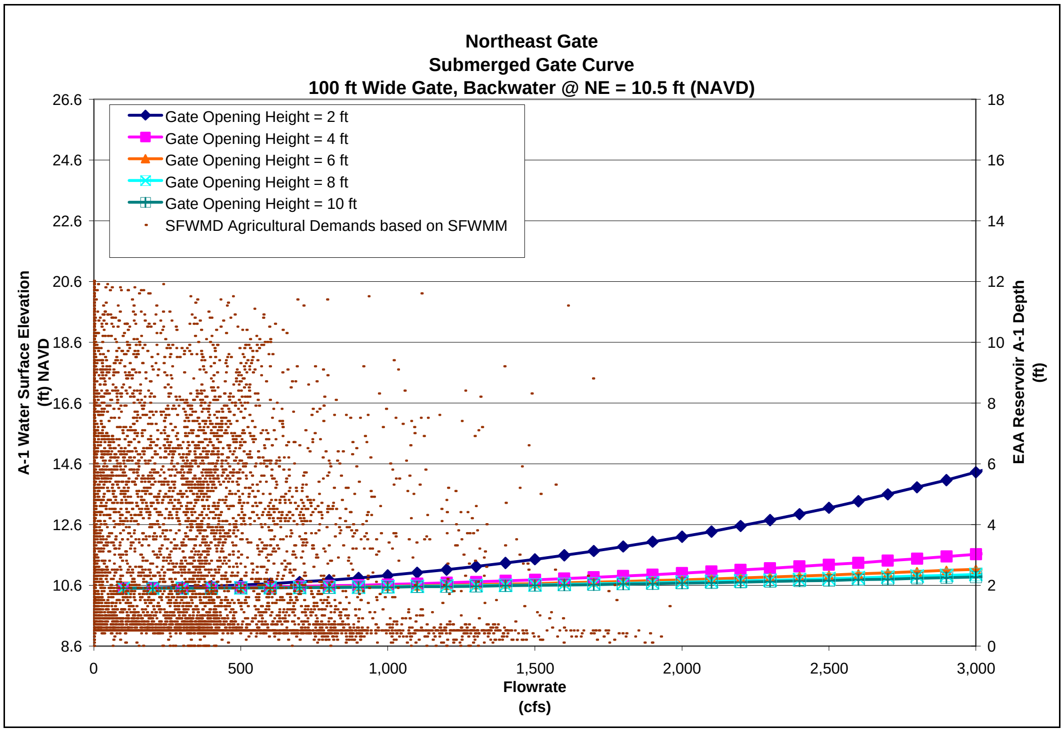


Figure 6.6-3 Northeast Gate Submerged Gate Curve 100-Foot Wide



6.6.1.2 Southeast and Southwest Gate Structures

The proposed southeast and southwest gate structures, as seen in Figure 6.6-1, will be bidirectional. They will be used to fill the EAA Reservoir A-1 and release water to the STA-3/4 Supply Canal for environmental deliveries. Because the G-383 gate structure, located in the STA-3/4 Supply Canal, separates the eastern STA-3/4 flow-way from the two western flow-ways, EAA Reservoir A-1 gate structures are required to seal in both directions. Since the Inflow Canal will be used to fill the EAA Reservoir A-1, the gates should have an inflow capacity to the EAA Reservoir A-1 equaling that of the pump stations' maximum capacities. G-370 and G-372 pump stations have a maximum pumping rate of 2,340 cfs and 3,120 cfs respectively when pumping to an EAA Reservoir A-1 water depth of 8 feet. The gates will be sized to pass their design flows with approximately 0.5 feet of head loss.

The southeast and southwest gate structures should also be capable of discharging the 30 percent PMP storm preceding a PMP event. To do this, they must be able to release 4,000 cfs to STA-3/4 for three days (see Section 5.1 for discussion). Since the head differential between the EAA Reservoir A-1 and the STA-3/4 Supply Canal will generally be greater than 0.5 feet, this drawdown requirement will be achievable.

The culvert/gate structures being considered for the EAA Reservoir A-1 are 10 feet x 10 feet culverts with roller gates on the external embankment. The gates will be submerged (i.e. orifice flow); with inverts located 10 feet below the bottom of the EAA Reservoir A-1, or approximately at -1.4 NAVD88. If a 50-foot gate is needed, then five parallel 10 feet x 10 feet culvert/gate structures will be placed in that location. To meet the inflow requirements of the southern gates and proposed design head 0.5 feet, the southeast gate will need to be 50 feet in width (five parallel 10 feet x 10 feet culverts/gates), while the southwest gate will need to be 70 feet in width

(7 parallel 10 feet x 10 feet culverts/gates). Figures 6.6-4, 6.6-5, and 6.6-6 show varying gate sizes with their corresponding flow rates at different head potentials across the gates. These figures were used to select the gate sizes discussed in this Section.

Figure 6.6-4 South Gate Flow Rate 30-Foot Gate

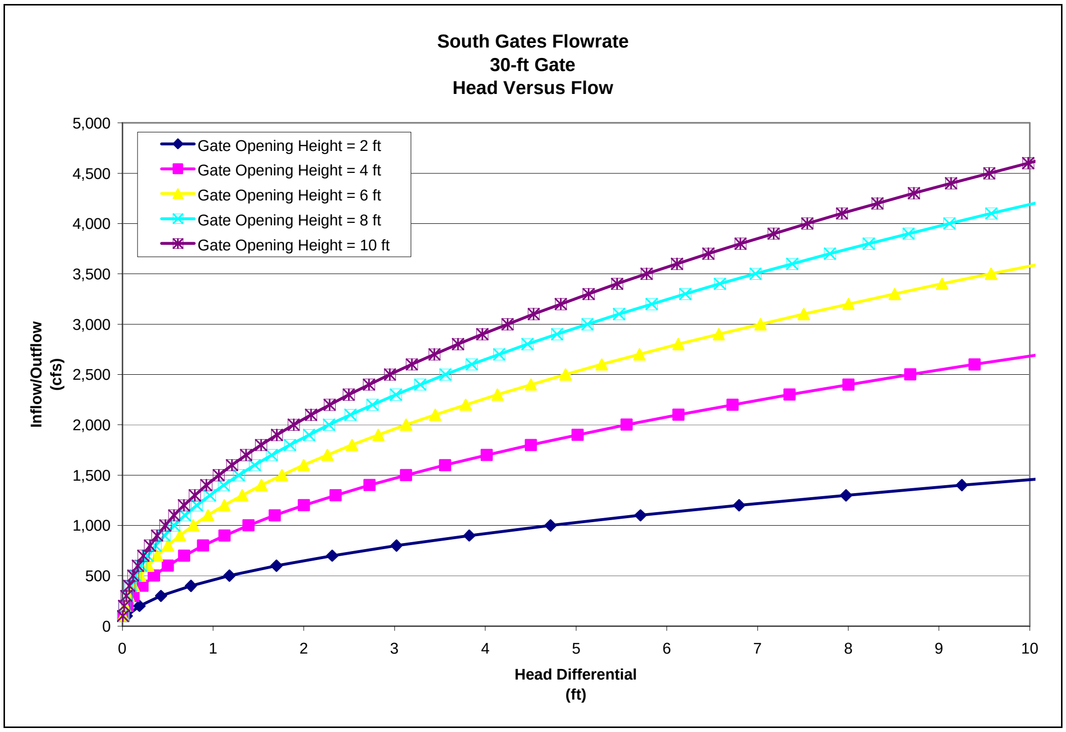


Figure 6.6-5 South Gate Flow Rate 50-foot Gate

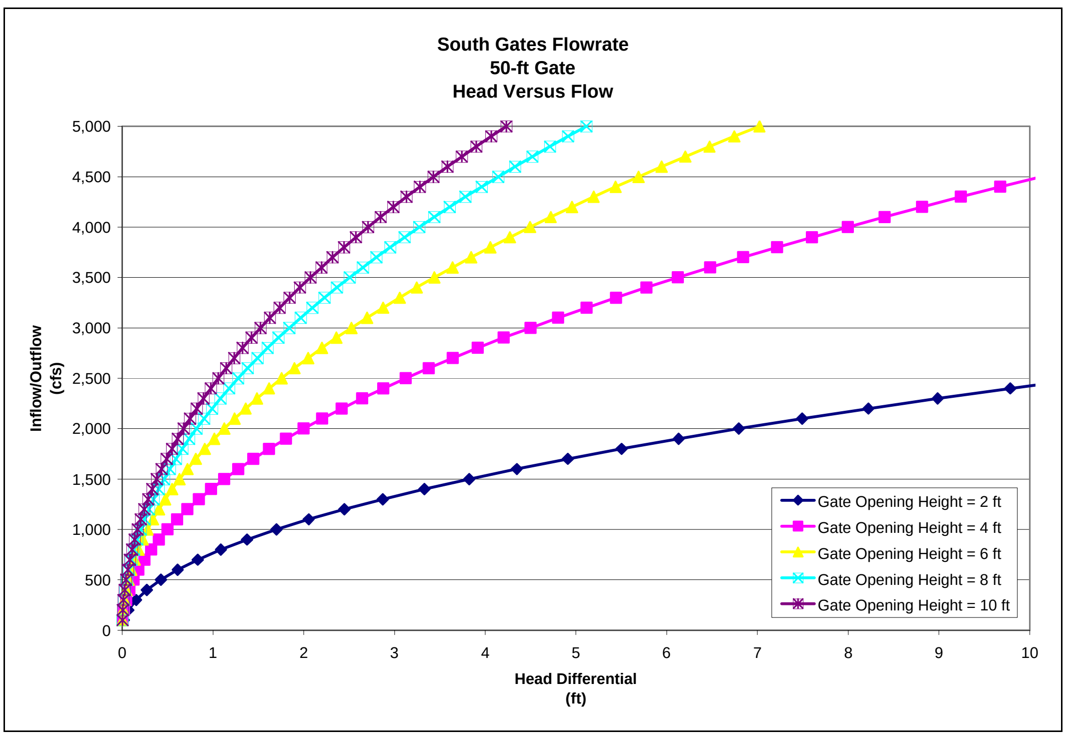
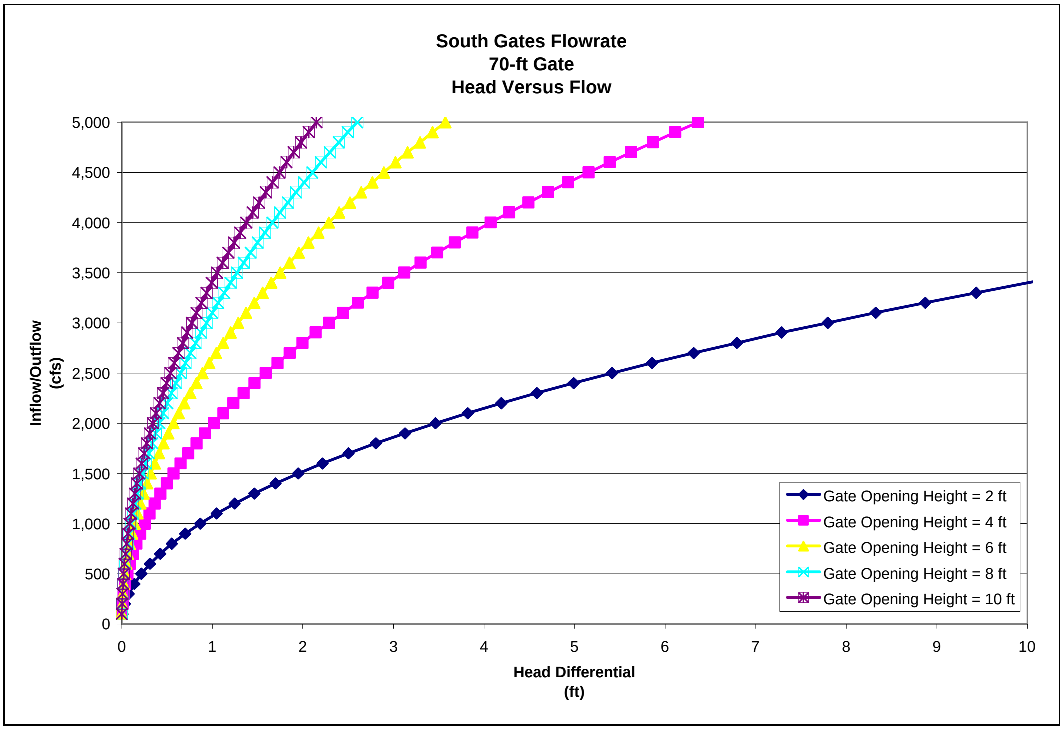


Figure 6.6-6 South Gate Flow Rate 70-foot Gate



6.7 SPILLWAY

6.7.1 General

For Acceler8 projects, DCM-3 establishes requirements for spillway capacity and EAA Reservoir A-1 drawdown. The DCM has a requirement to provide an uncontrolled spillway crest to ensure that the EAA Reservoir A-1 cannot be overfilled or maintained at a level above the normal full storage level (NFSL) (ref. Basis of Review for Environmental Resource Permit Applications within SFWMD, Appendix 6, "2.1.2.5 Return overflow … A separate structure will be necessary for pump filled impoundments to allow return flow under conditions of maximum or design water levels in the EAA Reservoir A-1 with pumps continuing to operate"). In reality, for EAA Reservoir A-1 the pumps will be designed with a shut off head that physically limits their capability to fill the EAA Reservoir A-1 very far beyond NFSL, and will not be able to pump to embankment crest level.

An uncontrolled spillway has the potential to reduce the freeboard allowance and thus embankment crest height and cost through routing of flood events. Spillway options for reducing embankment height were analyzed and are summarized herein.

6.7.2 Spillway operation

6.7.2.1 Operating Conditions

A range of operating conditions must be considered for a spillway:

- Storms up to 100-year return period, which is a cut-off for SFWMD's discharge limits
- Storms which are locally very intense (>100-year event) but which do not cause excessive rainfall outside the 25 square mile surface area of the EAA Reservoir A-1
- Storms more severe than 100-year return period but less severe than PMP
- PMP
- PMP plus wind and waves (possibly causing up to about 11 feet depth of water above NFSL)
- Wind up to PMW intensity, blowing towards the spillway with EAA Reservoir A-1 full (NFSL)

These conditions are described in the following sections with an assessment of the potential performance of uncontrolled spillways under these conditions.

6.7.2.2 Discharge Limit up to 100-Year Storm Intensity

DCM-3 limits the off-site discharges for storm events up to 100-year return period but does not limit discharge for events of lower probability. The hydrograph of the three day 100-year storm is shown in Figure 6.7-1.

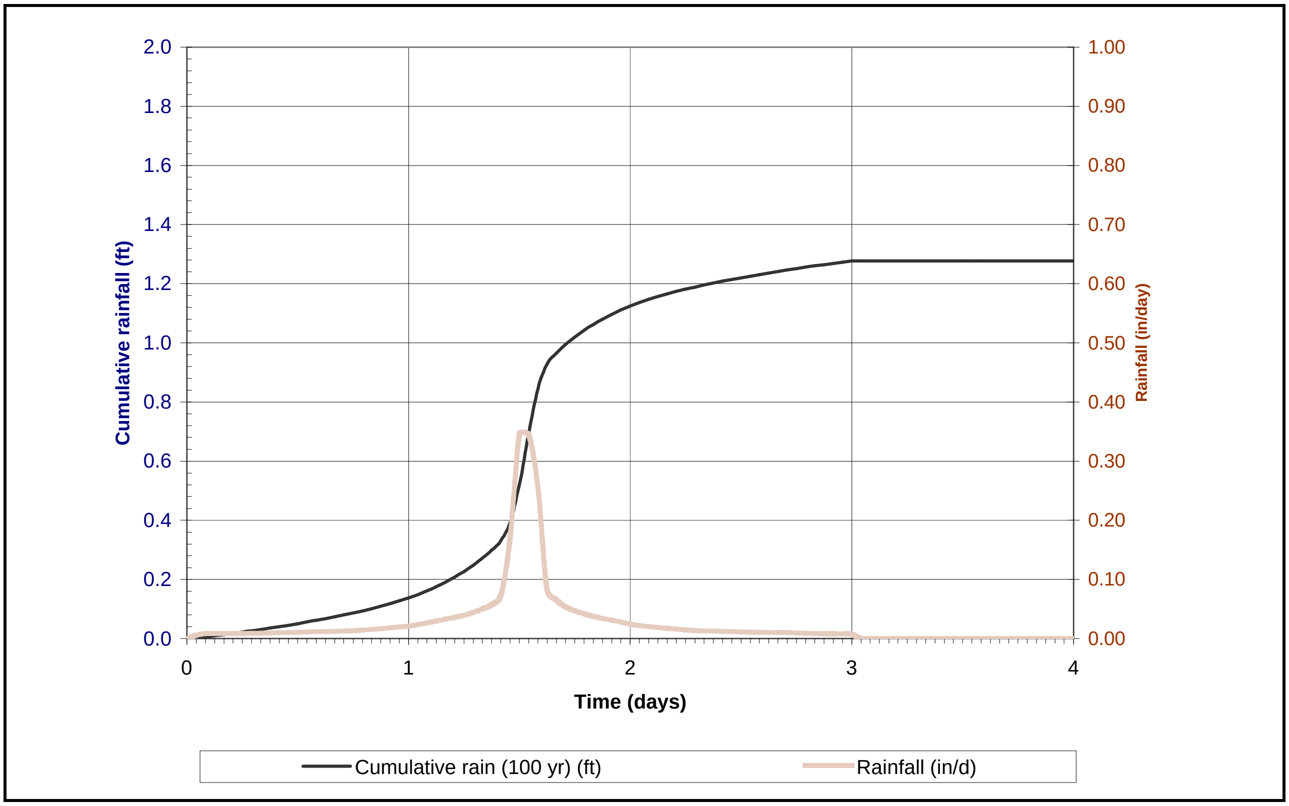
SFWMD Basis of Review for Environmental Resource Permit Applications (BOR), Section 6.2, establishes the following rules for discharge. Any spillway option for reducing embankment height would need to meet all of these conditions.

> Off-site discharge rate shall be limited to:
>
> (a) Rates not causing adverse impacts to existing off-site properties, and
>
> (b) Historic discharge rates, or
>
> (c) Rates determined in previous SFWMD permit actions, or
>
> (d) Rates specified in SFWMD criteria (see BOR Appendix 2)

Previous permit actions are consistent with the current SFWMD criteria as outlined in Appendix 2 of the BOR.

For the EAA, the SFWMD criteria allow a discharge of 20 cfs per square mile with a five year design frequency (equal to 3/4 inch of runoff per 24 hours). For the EAA Reservoir A-1 area this equates to 500 cfs discharge maximum. If this limit is exceeded, it reduces the canal capacity available to other discharges. Therefore, this criterion must be met.

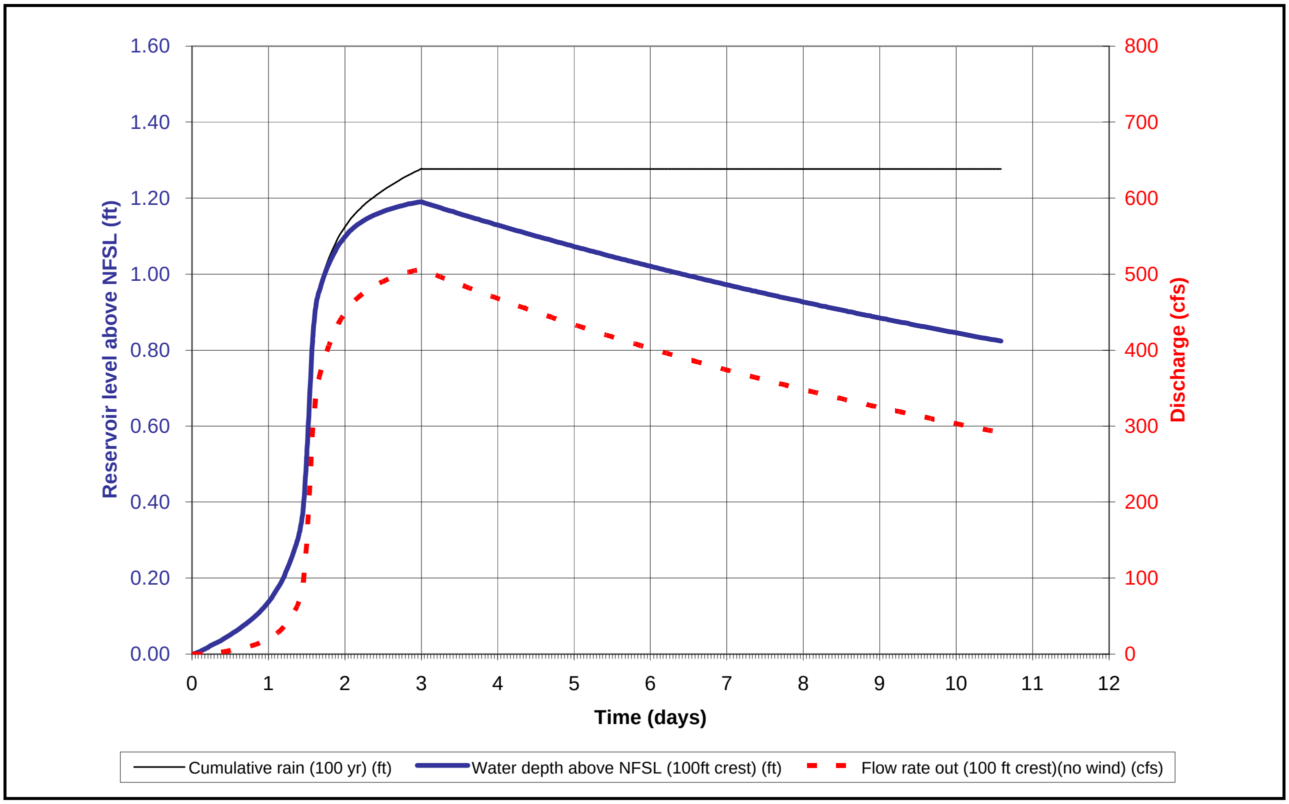
Figure 6.7-1 Three Day 100-Year Storm Hydrograph



In order to maximize the potential flood level reduction available from a spillway, discharge should begin at the earliest possible time during a storm, at the maximum rate possible.

Assuming an ogee-type weir with discharge coefficient of 3.9, the spillway crest width at NFSL is limited to 100 feet to meet a maximum discharge of 500 cfs. This is shown in the flood routing graph, Figure 6.7-2. The 3.9 discharge coefficient represents the most efficient spillway operation that can be achieved under these conditions and has been assumed for all of the spillway analyses.

Figure 6.7-2 72-Hour 100-Year Storm Routing over a 100 foot Long Spillway



During this event, with a 100-foot spillway crest, the water rises approximately 1.2 feet above NFSL. Therefore, in accordance with DCM-3, which specifies no limit to discharge above the 100-year event, the spillway length could be increased at or above that level to allow more discharge. This scenario is examined below.

6.7.2.3 Storms More Severe Than 100-Year Return Period but Less Severe Than PMP

DCM-3 indicates that, based on the common practice in the U.S., discharges from reservoirs should not increase downstream flows for events up to a 100-year annual return interval (ARI). The DCM allows unlimited discharge beyond the 100-year storm event.

A 100-year event would be 15.2 inches of precipitation over a 72-hour period. Under these or worse conditions, the SFWMD and agricultural users canal/pump station conveyance system would be overwhelmed, resulting in localized flooding.

Potentially large, high velocity, concentrated discharges from an uncontrolled spillway are capable of damaging structures such as bridges and roads. In addition, uncontrolled general overtopping of conveyance canals can result in extensive damage to structures. Therefore, some means of control must still be designed into the spillway structure.

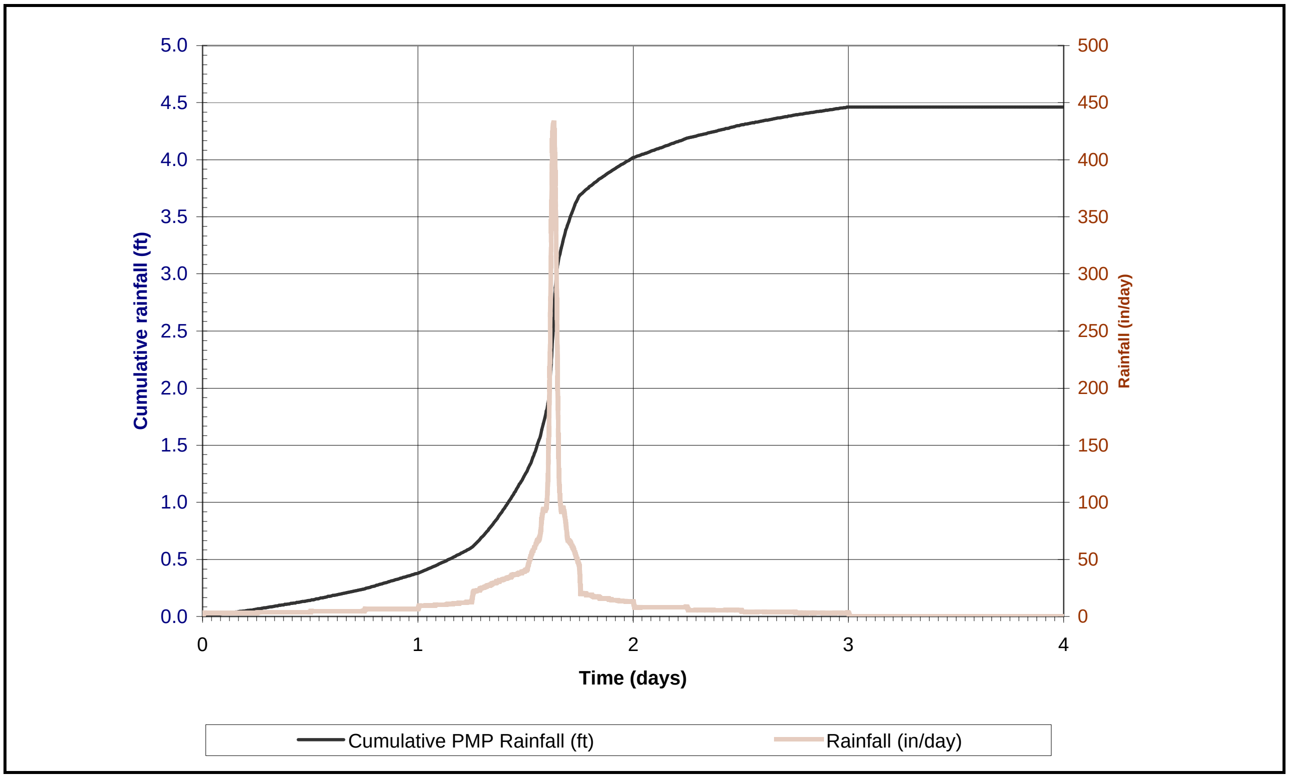
6.7.2.4 Storms which are Locally Very Intense (>100-Year Event) but which do Not Cause Excessive Rainfall Outside the 25-Square Mile Area of the EAA Reservoir A-1

Storm activity can be very isolated in the Everglades region. It is possible for an extreme storm cell to concentrate rainfall only over the EAA Reservoir A-1 (this is the same scenario required for the PMP event). Should the EAA Reservoir A-1 be full under such conditions, an uncontrolled spillway would discharge water in excess of the 500 cfs limit. This could adversely impact existing off-site properties.

6.7.2.5 Probable Maximum Precipitation (PMP)

The PMP is the most severe classification of storm used in EAA Reservoir A-1 design and is applied in this case because the structure has a high hazard potential classification (Section 5). The three day PMP event totals 54 inches of rainfall as shown on Figure 6.7-3. This total rainfall is based specifically on the EAA Reservoir A-1 area (point rainfall). If a wider area such as the EAA were considered the intensity would be about 42 inches in a 72 hour period.

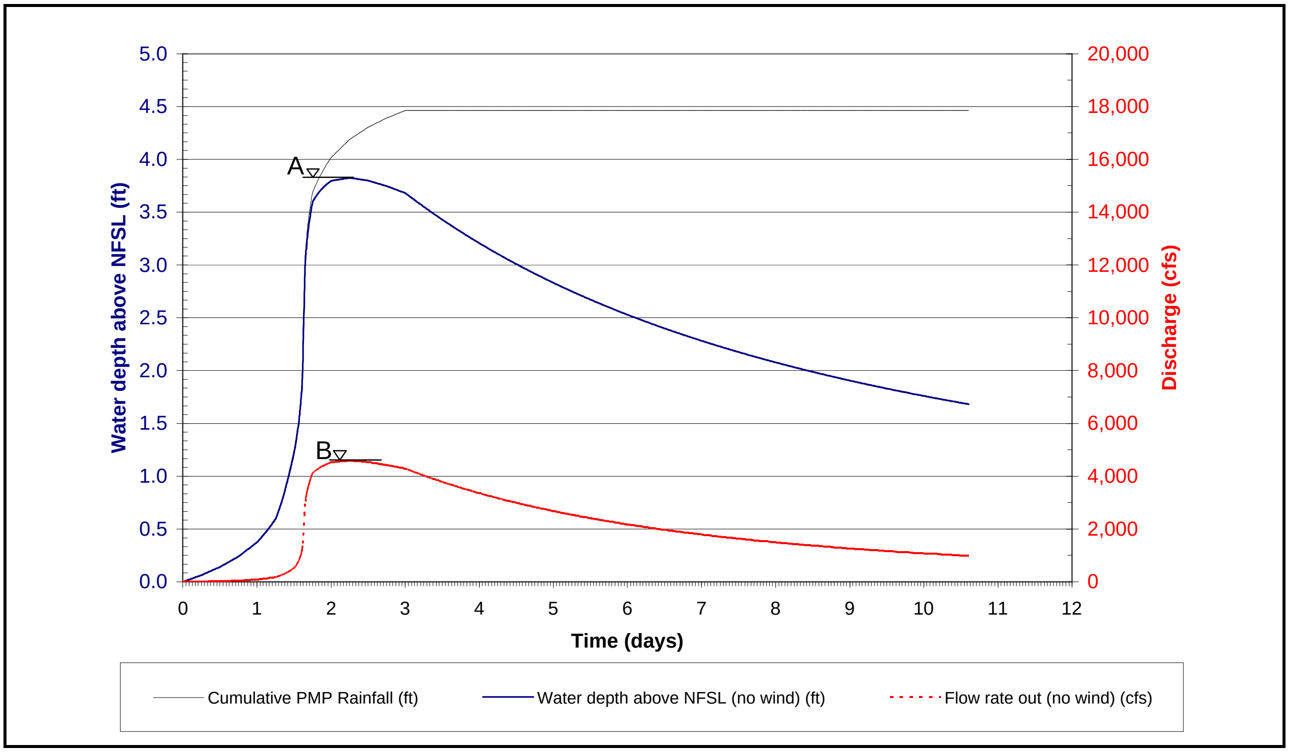
Figure 6.7-3 Probable Maximum Precipitation (PMP)



The peak EAA Reservoir A-1 rise caused by the PMP must be reduced by discharging water through a spillway during the rainfall event. To assess this effect, analysis included calculations based on uncontrolled releases over the top of a crest spillway. Assuming an ogee weir, the spillway crest length at NFSL is limited to 100 foot, which results in a EAA Reservoir A-1 rise of about 1.2 feet. Above that level a longer spillway can be incorporated.

As an example, the results from an analysis routing the PMP over a 200-foot stepped spillway (100 feet at NFSL and 100 feet at NFSL + 1.2 feet) are shown in Figure 6.7-4. If there was no discharge from the EAA Reservoir A-1 during the storm, the water level rise would be equal to the cumulative rainfall during the period. Point A on the graph represents the peak EAA Reservoir A-1 rise for the routed discharge and shows a maximum water level reduction of 0.64 feet for the 200-foot stepped spillway. Point B represents the peak discharge during the storm which is about 4,600 cfs.

Figure 6.7-4 Routing the PMP over a 200-Foot Stepped Spillway



6.7.2.6 PMP Plus Wind Away from the Spillway

Strong prevailing winds will cause water to surge against the side of the EAA Reservoir A-1 by two mechanisms: wind set-up and waves. Should wind be in the direction of the spillway, it will increase momentary discharge. Should wind be away from the spillway, it will decrease momentary discharge.

Wind set-up causes a general increase in the water elevation in the direction of the wind. Waves cause a fluctuating water depth. The combined effects of set-up and waves could result in peaks of as much as 11 foot depth of water above NFSL during the PMP as shown in Table 6.7-1. Wave run-up does not apply to flow over a spillway and is not included in these figures. When wind blows in the opposite direction water level is correspondingly reduced.

Table 6.7-1 Potential Wind Effects on Water Level above PMP

Wind event	Wind Speed (mph)	Wind Set-Up (feet)	Wave Height (feet)	Peak Water Level Above NFSL (feet)
100-year	103	2.1	6.6	9.9
Category five storm	122	3.6	6.5	11.3

DCM-3 specifically points out that the theoretical efficiency of a single emergency spillway will be limited because of sustained wind:

> *"Spillway design must consider that sustained winds could come from any direction. Sustained winds on long fetches can cause substantial wind setup during the IDF. If wind setup causes increased water levels on the windward side of the reservoir, it will also cause lower water levels on the leeward side. Reservoir routing of the IDF should consider the most critical effects of wind setup on water levels at the spillways. Wind setup requirements may require spillways at more than one location and/or opposite ends of the reservoir."*

For the EAA Reservoir A-1 it is impractical to have more than one spillway to counter-act this effect due to the limited directions of discharge as discussed in Section 6.7.3.

DCM-3 does not give specific guidance on the effects of wind so, to allow for this effect, it was considered that an adverse wind might direct surges away from the spillway reducing the water level at the discharge. A 12-hour period was considered, at the peak of the three day PMP storm, with a 103-mph wind.

The effect of wind blowing away from the spillway on the routing of the PMP flood over a 200 foot long crest is shown in Figure 6.7-5. Initially, when the wind is blowing, discharge from the spillway is reduced. The wind delays releases, which allows the water level to rise higher than the no-wind case. The peak discharge occurs when the wind drops and because water level is increased, peak discharge is also increased. Wind blowing away from the spillway increases and delays the peak of the hydrograph from point B to point D on the graph. The efficiency of the spillway to reduce the EAA Reservoir A-1 rise is decreased as illustrated by comparing points A (without wind) and C (with wind).

The results of similar analyses carried out for different spillway lengths from 0 to 400 feet long are illustrated in Figure 6.7-6. The four colored lines on the graph correspond to the peaks of discharge and EAA Reservoir A-1 rise, both with and without wind effects. The results for a 200-foot long spillway, described earlier, are illustrated and the points A, B, C, and D correspond to the points on Figure 6.7-5. These results are for stepped spillways for lengths above 100 feet.

The results show that a 400-foot long stepped spillway would allow an embankment crest height reduction of only 0.6 feet. The peak discharge rate with wind blowing away from the spillway would be approximately 8,000 cfs.

Figure 6.7-5 Routing of PMP over a 200-Foot Stepped Spillway Allowing for Wind Effects

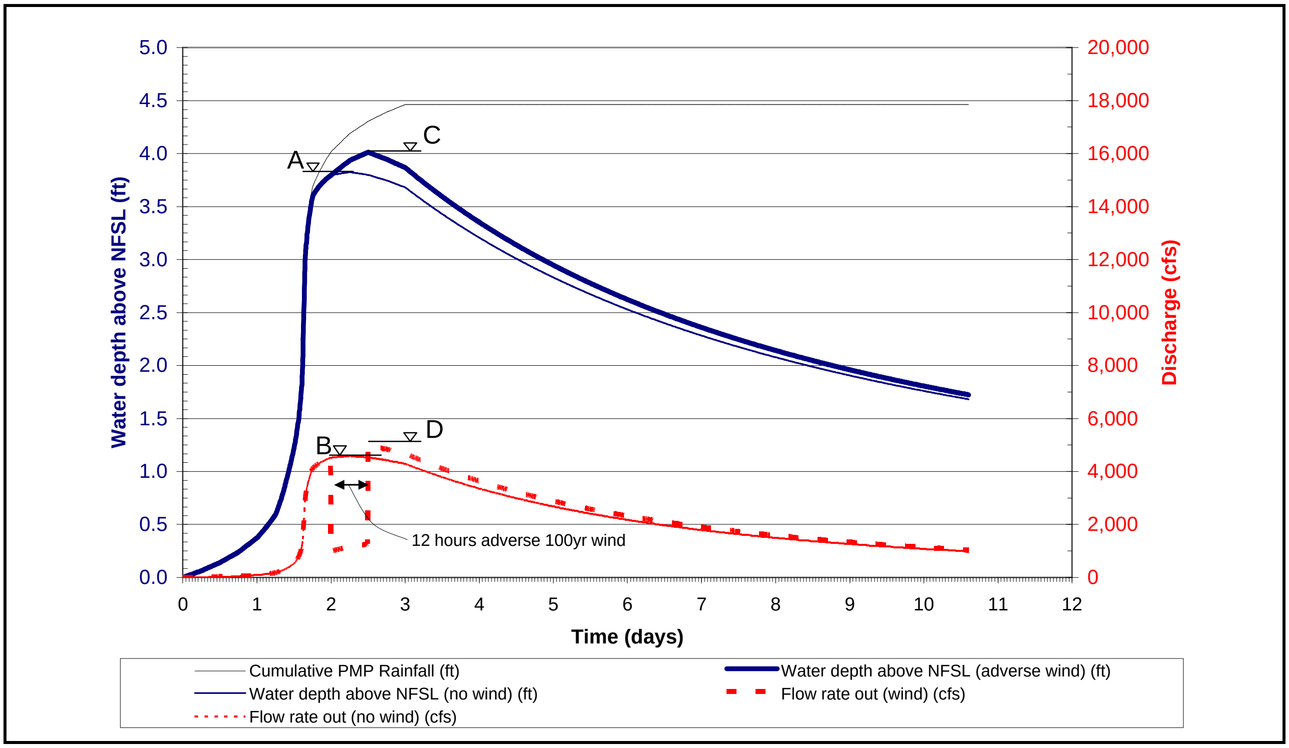
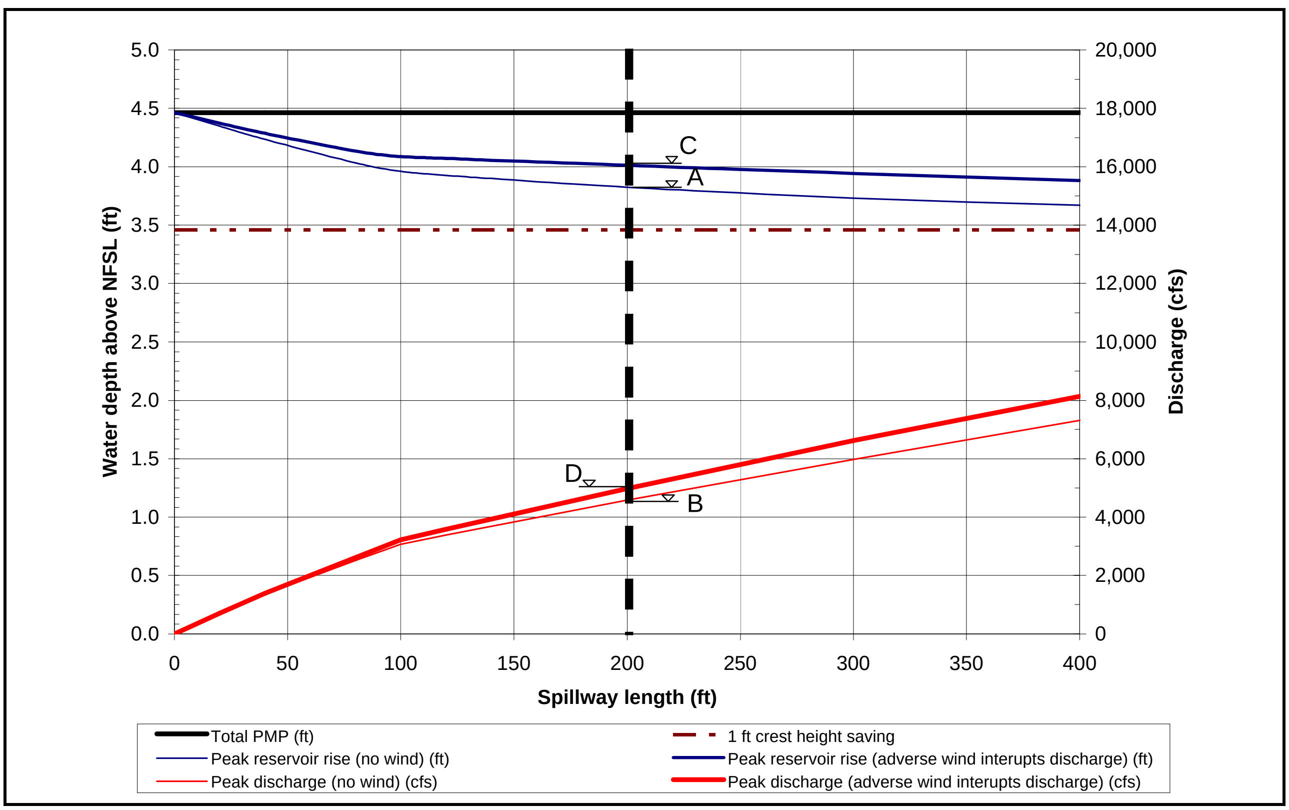


Figure 6.7-6 Flood Routing for Spillways up to 400 Feet Long

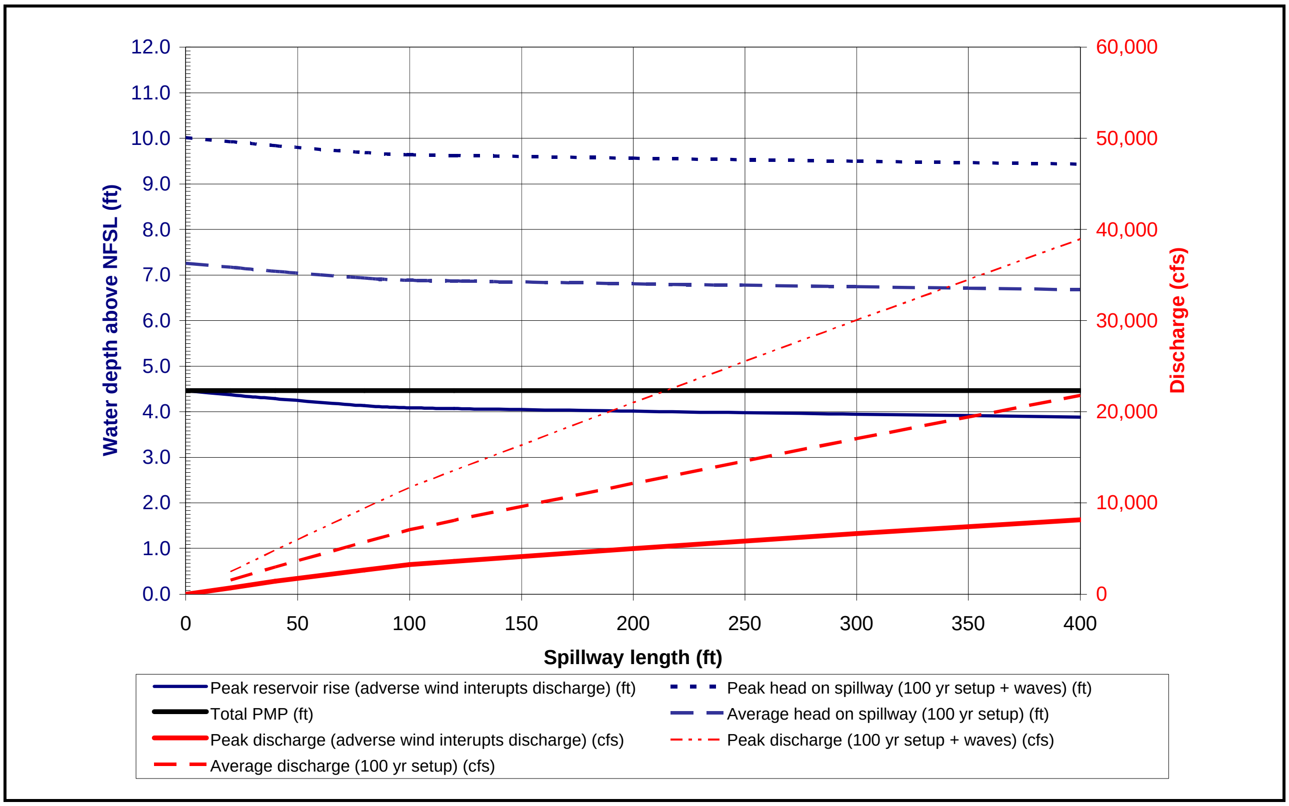


6.7.2.7 PMP Plus Wind Toward The Spillway

Wind could blow towards the spillway during the PMP event. This would cause surge adjacent to the spillway increasing discharge. Two effects have been considered: wind set-up and wave action. The new discharge would be based on the EAA Reservoir A-1 rise due to rainfall (reduced allowing for routing effects), plus wind set-up. Waves cause an oscillation of water level above and below this, so the peak wave level would include the wind set-up allowance plus half the wave height. The average condition would just include set-up. These set-up and wave height values are given in Table 6.7-1.

The results of the analyses are shown in Figure 6.7-7. For all spillway lengths between 0 and 400 feet the peak water depth over the spillway is between 9.5 and 10 feet. The peak discharge over a 400-foot long spillway would reach nearly 40,000 cfs; average flows of 22,000 cfs.

Figure 6.7-7 Effects of PMP Plus Wind



The DCM states that these flows are acceptable given that the EAA and areas downstream would already be flooded during such an event. However, although the PMP for the EAA Reservoir A-1 is rated as 54 inches of precipitation, this would not be the average level in the EAA; the regional average could be considerably less. The reality of potentially large, high velocity, concentrated discharges is very different conceptually to a few feet of relatively passive flood water. Such fast moving flows are capable of damaging structures such as bridges and highways. U.S. 27, a major hurricane evacuation route, is immediately adjacent to the EAA Reservoir A-1. Damage to the road could limit evacuation of southern Florida.

6.7.2.8 Reservoir Full (NFSL) with Wind Blowing Towards the Spillway

Winds will cause water to surge against the side of the EAA Reservoir A-1 even when there is no rainfall in the area. With the EAA Reservoir A-1 full, water at NFSL, the conditions shown in Table 6.7-2 can be expected. Wave run-up does not apply to this case and is not included in these figures.

The wind effects discussed in Section 6.7.2.7 could occur even when there is no rain, but also when the EAA Reservoir A-1 level is maintained at NFSL operationally. The effects of wind set-up are increased at shallow water depths but the wave heights are reduced.

Table 6.7-2 Potential Wind Effects on Water Level above NFSL

Wind Event	Wind Speed (mph)	Wind Set-Up (feet)	Wave Height (feet)	Peak Water Depth above NFSL (feet)
100-year	103	2.8	5.5	5.5
500-year	119	3.8	6.0	6.8
Category five storm	122	4.0	6.1	7.0
PMW	158	7.0	7.1	10.5

Discharges resulting from analysis of the wind effects are illustrated in Figure 6.7-8. In the figure, the 500 cfs discharge limit appears as a red line right at the base of the graph. By comparison, some of the potential discharges from an uncontrolled crest spillway are much larger. For example, the peak discharge from a 200-foot long spillway could reach 25,000 cfs during a probable maximum wind event. The same structure could release an average flow of approximately 5,000 cfs if a category five hurricane made landfall in the area.

Figure 6.7-8 Potential Dry Weather Discharges Due to Wind

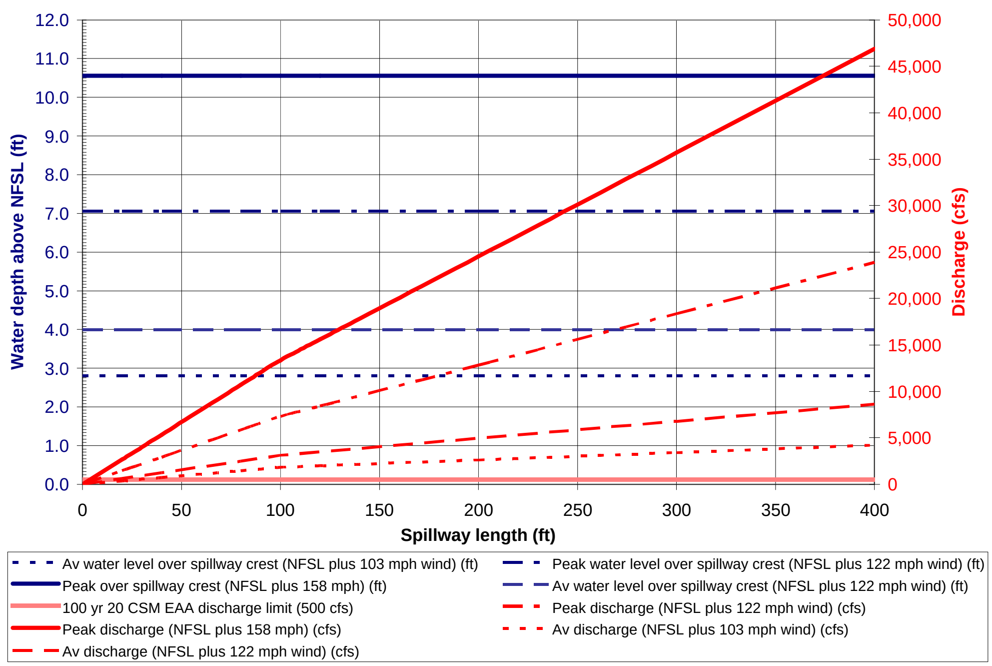
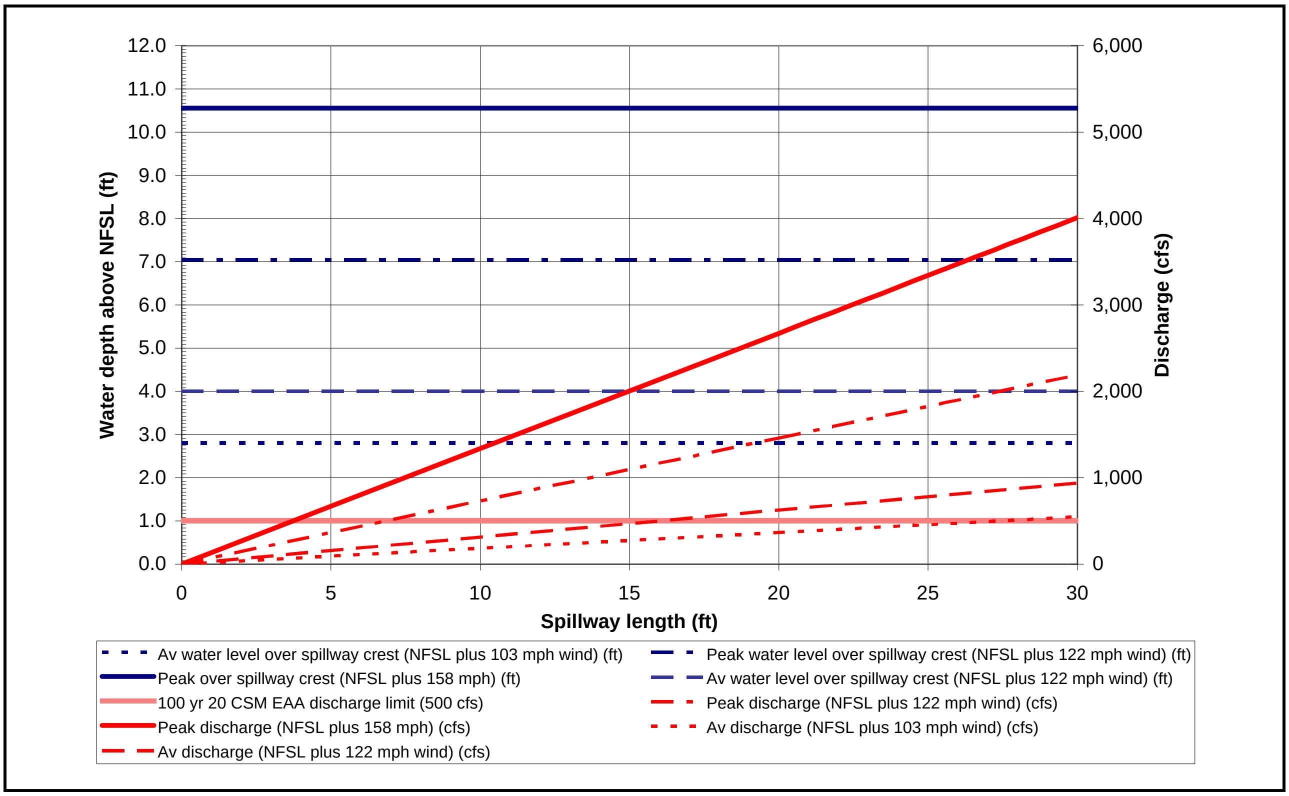


Figure 6.7-9 shows the dry weather wind generated discharges in more detail. Under a 103-mph wind scenario, a spillway length of about 25 feet maximum would be necessary to limit average discharges to 500 cfs. Considering a 122-mph wind, as could be generated by a category five hurricane making landfall in the area, this spillway length would need to be reduced to about 15 feet. In either of these situations peak discharge would exceed the 500 cfs limit.

As discussed in Section 6.7.3.7, large discharges during a PMP event may be considered acceptable by some because the EAA could already be flooded. However, the potential damage caused by such flows when there is no associated extreme rain event would be unacceptable. If there is no associated rain event more extreme than the 100-year storm, the SFWMD rules clearly require that discharges be restricted to 500 cfs or less.

Figure 6.7-9 Wind Generated Flow Limit to Spillway Size



6.7.3 Location of Discharge

SFWMD must be sensitive to the needs of the stakeholders adjacent to the EAA Reservoir A-1 so the constraints around the perimeter must be considered for potentially large uncontrolled spillway discharges.

6.7.3.1 Private Property to the North and Northwest of the EAA Reservoir A-1

Unless captured and rerouted by a canal, uncontrolled discharges toward the north of the EAA Reservoir A-1 would not be acceptable. Without a canal sufficiently sized for all discharges, water allowed to discharge in an uncontrolled manner could cause a negative impact on the farmland.

With the existing canal system, water collected and routed elsewhere could only be directed to the NNRC. The restrictions applying to the NNRC are described below. This water would need to be pumped into the NNRC.

6.7.3.2 Agricultural Lands to the West of EAA Reservoir A-1

The agricultural land to the west of EAA Reservoir A-1 is to be leased for agriculture until EAA Reservoir A-2 is constructed. Uncontrolled discharges to this area would flow into, and could overwhelm, the agricultural canal system. Such discharges could occur at any time there was a rain event with the EAA Reservoir at NFSL or when the wind blew from the east causing wind set-up above NFSL. These discharges would need to be directed to the Miami Canal. In order to make a significant difference to the freeboard allowance on EAA Reservoir A-1, very large quantities of water would need to be released requiring a substantial spillway and water conveyance system. This would be a temporary alternative because it would be abandoned after EAA Reservoir A-2 was built.

6.7.3.3 Holey Land

The Holey Land is separated from EAA Reservoir A-1 by the STA-3/4 Supply Canal. For water to be conveyed from the EAA Reservoir A-1 to the Holey Land, flow would need to pass under, over, or through the Supply Canal. The Holey Land is potentially an area that could accept emergency storm flows.

During a storm event it is likely that both G-370 and G-372 pump stations will pump water into the Supply Canal. The discharge of large volumes through the Supply Canal would risk overtopping the embankment. Solutions to pass substantial flows over or under the canal would be very costly.

6.7.3.4 STA-3/4

Uncontrolled storm discharges could be routed directly to STA-3/4. However, the same practical problems of routing flow to the other side of the Supply Canal as for the Holey Land apply. In addition, SFWMD plans to operate STA-3/4 in a controlled manner treating agricultural runoff pumped through G-370 or G-372 pump stations, or controlled releases from EAA Reservoir A-1.

Being an 'off-site discharge', DCM-3 requires the same limit for EAA Reservoir A-1 runoff discharges to STA-3/4 as it would to the NNRC, i.e. 500 cfs for anything less than the 100-year storm. This would limit the crest length of an uncontrolled spillway placed at NFSL to 100 feet and would allow up to 500 cfs to discharge to the STA during the 100-year storm. STA-3/4 is designed for flows up to 6,000 cfs; so more water might be discharged if allowed by the SFWMD. However, higher discharge rates from the EAA Reservoir A-1 would limit the amount of water that could be conveyed through the Supply Canal and treated in STA-3/4 from NNRC and the Miami Canal.

Under certain conditions, uncontrolled discharges would be higher than the design capacity of the STA-3/4. Under these conditions the STA-3/4 could be damaged and might take as much as 12 to 18 months to recover.

6.7.3.5 NNRC, U.S. 27 and Properties East of the EAA Reservoir A-1

The NNRC and pump stations (S-2 and S-7) are sized to convey approximately 20 cfs per square mile from their contributing watersheds. All inflows to NNRC are by pumping so agricultural discharges are controlled by installed capacity. If too much is pumped into the canal system, the water, in essence, backs up and floods other farms in the basin because the NNRC system cannot remove it fast enough.

The assurances require that the EAA Reservoir A-1 Project does not adversely affect the flood protection level of the farmers. This implies that flood discharges to the NNRC from the EAA Reservoir A-1 should be capped at 20 cfs per square mile up to a 100-year storm.

6.7.4 Economic Assessment

Two conceptual spillway layouts were developed as a basis for a cost analysis. Each structure is based on an ogee shape but, one spillway would be a completely uncontrolled and one would have some form of gated protection on the crest. Conceptual level arrangements are shown on Figure 6.7-10 and 6.7-11. The probable costs of 200-foot long spillways were calculated using take-off quantities and unit rates, modified as appropriate. That cost was then used as the basis for costing spillways of lengths between 0 and 400 feet. In recognition of the fact that cost per foot would decrease for longer structures and conversely increase for shorter structures, a 0.875 power function was used in lieu of linear interpolation and extrapolation.

Figure 6.7-12 summarizes the cost analysis and Table 6.7-3 shows more detailed figures. In the Figure the red lines represent embankment cost savings, both due to height reduction and length replaced by spillway. The upper blue lines represent the additional cost of the spillway. The purple lines show the net additional cost, which is always positive in this case, i.e. provision of a spillway does not save money.

Figure 6.7-10 Conceptual Uncontrolled Ogee Crest Spillway

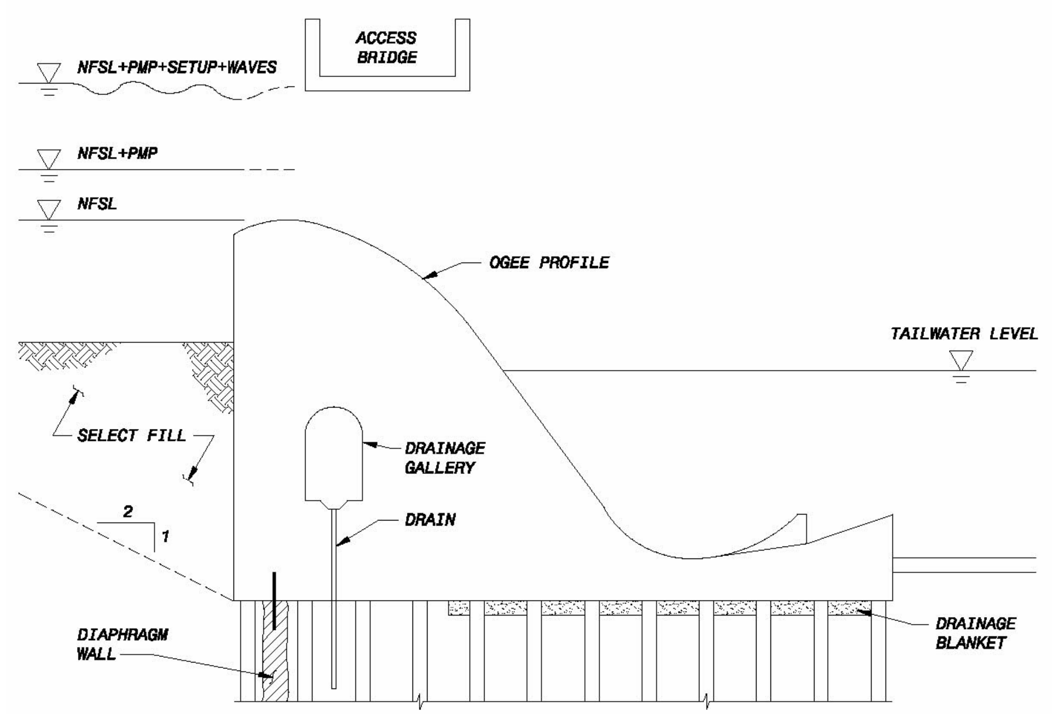


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Figure 6.7-11 Conceptual Controlled Ogee Crest Spillway

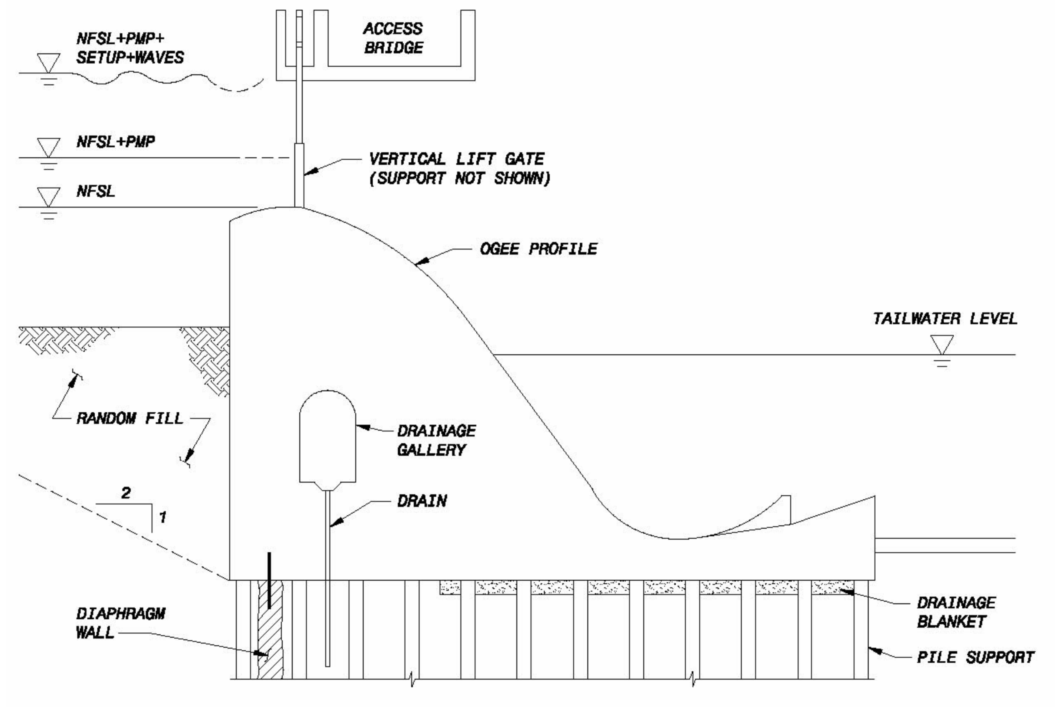

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Figure 6.7-12 Economic Assessment of Spillway Alternatives

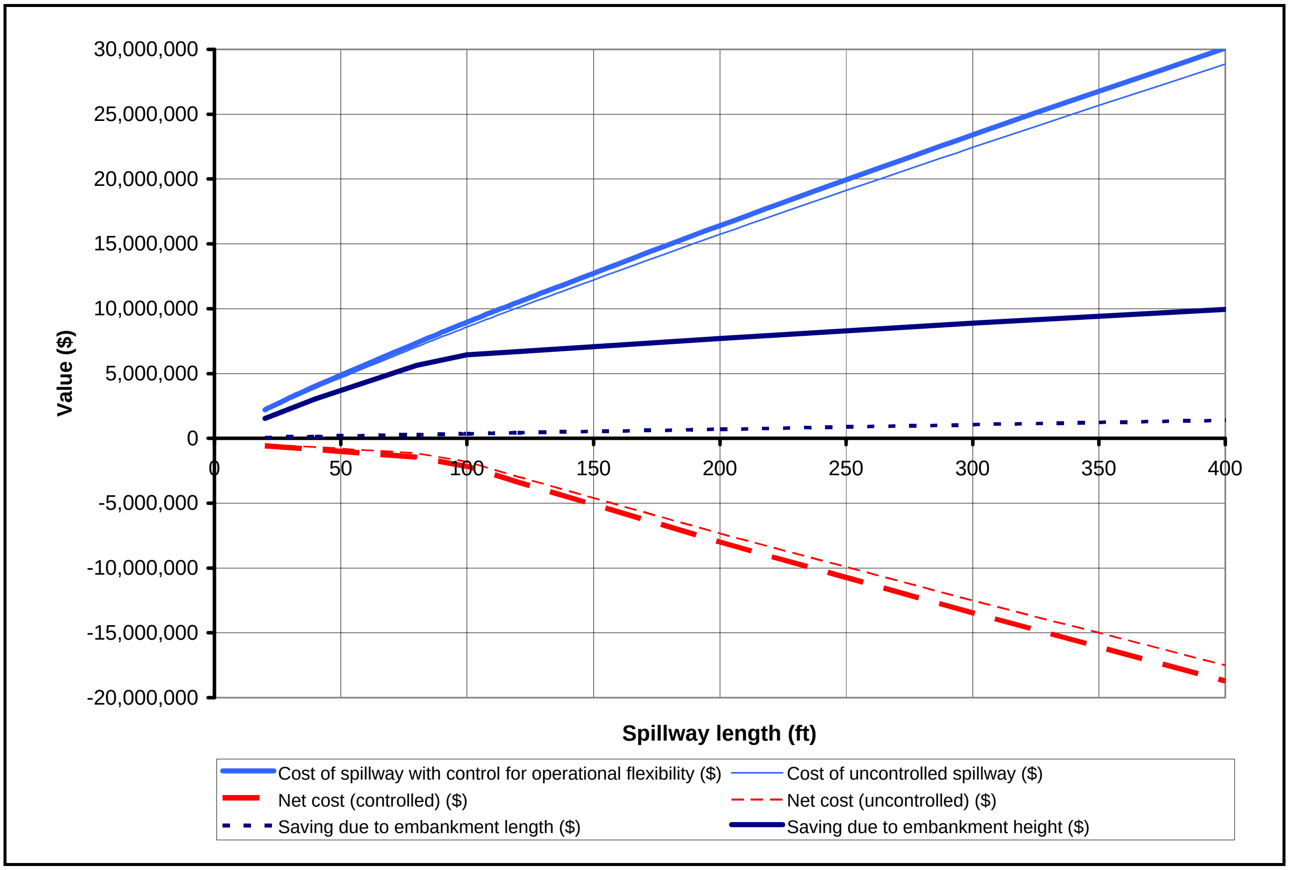

Table 6.7-3 Economic Evaluation of Spillway Options

		Cost for 200 Feet Long ($)	Cost for 200 Feet Long ($)	Saving ($ per Linear Foot)	Saving ($ per Foot Height)		
		15,700,000	16,400,000	3,500	17,100,000		
Spillway Length (feet)	**Possible Crest Height Reduction** (Feet)	**Cost of Uncontrolled Spillway** ($)	**Cost of Spillway with Control for Operational Flexibility** ($)	**Saving Embankment Length Cost** ($)	**Saving Embankment Height Cost** ($)	**Net Effect (uncontrolled)** ($)	**Net Effect (controlled)** ($)
0							
20	0.09	2,100,000	2,200,000	(70,000)	(1,500,000)	500,000	600,000
40	0.18	3,800,000	4,000,000	(140,000)	(3,000,000)	700,000	900,000
80	0.33	7,100,000	7,400,000	(280,000)	(5,600,000)	1,200,000	1,500,000
100	0.37	8,600,000	8,900,000	(350,000)	(6,400,000)	1,800,000	2,200,000
120	0.39	10,100,000	10,500,000	(420,000)	(6,700,000)	3,000,000	3,400,000
200	0.45	15,700,000	16,400,000	(700,000)	(7,700,000)	7,300,000	8,000,000
300	0.52	22,400,000	23,400,000	(1,000,000)	(8,900,000)	12,500,000	13,500,000
400	0.58	28,800,000	30,100,000	(1,400,000)	(10,000,000)	17,500,000	18,700,000

6.7.5 Recommendations

6.7.5.1 Spillway Location

The discharge capacity at NFSL should allow the water level in the EAA Reservoir A-1 to naturally return to that level. There are two alternatives for locating such a structure, discharging either into the NNRC at the northeast pump station or into the STA-3/4 Supply Canal. If the discharge is to the NNRC, it will utilize some canal capacity which the agricultural users would otherwise have available for flood protection. In addition, water discharged to the NNRC could be pumped back to STA-3/4 by G-370 pump station.

If discharge is to the Supply Canal there may be operational objections to having some STA-3/4 capacity used up by uncontrolled flows from the reservoir. This could also be an operational problem because gates would need to be open from the Supply Canal to STA-3/4. Under these conditions, releasing back to the NNRC would be preferred.

The recommended location is at the northeast pump station discharging to the NNRC. The spillway can be incorporated into the headwalls and aprons of the proposed structure.

6.7.5.2 Spillway Configuration

We recommend that an orifice spillway be constructed and the flow limited to 500 cfs at 2 foot of head by constricting the discharge conduit. This will ensure that discharge at the 100-year rainfall level will be less than the 500 cfs limit and discharge will not be destructive at more extreme events. There is adequate storage to temporarily handle any excess rainfall from high frequency short-term events.

For a 500 cfs release rate at 2 feet of head over a weir, a weir length of approximately 55 feet will be required. A box culvert approximately 5.5 x 5.5 feet would act as an orifice to limit the flows to reasonable limits. The spillway configuration will be confirmed during the next phase of design.

6.8 SEEPAGE PUMPS

6.8.1 Northeast Pump Station

As discussed in detail in Sections 9 and 10 hereafter, seepage collection canals will be constructed along portions of the western boundary, the northern boundary, and the eastern boundary of the EAA Reservoir A-1 site. The collected seepage will be directed to the new northeast pump station via those canals. At the northeast pump station seepage pumps will be selected to provide firm capacity to pump seepage back into the EAA Reservoir A-1 via four seepage pumps. In order to provide the firm capacity of 250 cfs as modeled for the selected seepage collection alternative in Section 9, four 85-cfs seepage pumps will be included at the northeast pump station.

6.8.2 G-370 Pump Station

Since the recommended embankment along the STA-3/4 Supply Canal will require the filling of the existing seepage canal on the north side of that canal, the seepage pumps located at G-370 pump station will no longer be necessary to serve that seepage collection canal. Therefore, the G-370 pump station seepage pumps will be connected to the EAA Reservoir A-1 seepage collection canal in the southeast corner of the site. The existing G-370 pump station seepage pumps will then serve the EAA Reservoir A-1 seepage canal when one or more pumps are out of service at the northeast pump station or when a higher pumping rate is necessary to maintain seepage canal levels.

Provisions will need to be made during construction of the EAA Reservoir A-1 for handling seepage from the north side of the STA-3/4 Supply Canal as the seepage canal is filled with embankment materials. Temporary channels will be required to carry seepage to the G-370 pump station seepage pumps to prevent flooding of the agricultural area remaining within the EAA Reservoir A-1 area during construction. The sequence of construction established for the construction contractor will determine the steps required.

6.8.3 G-372 Pump Station

The existing seepage pumps located at the G-372 pump station will continue to serve the seepage collection canal along the north side of the STA-3/4 Supply Canal where that canal runs along the Holey Land Wildlife Management Area. During detailed design, consideration will be given to connecting that seepage canal to the EAA Reservoir A-1 seepage collection canal at its end along the west border via a gated structure. If this connection is selected, the seepage pumps at the G-372 pump station would also be able to serve the EAA Reservoir A-1 seepage canal.

6.9 REFERENCES

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SECTION 7

SUBSURFACE CONSIDERATIONS FOR CONSTRUCTION

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FIGURES

7. SUBSURFACE CONSIDERATIONS FOR CONSTRUCTION

7.1 GEOTECHNICAL EXPLORATION

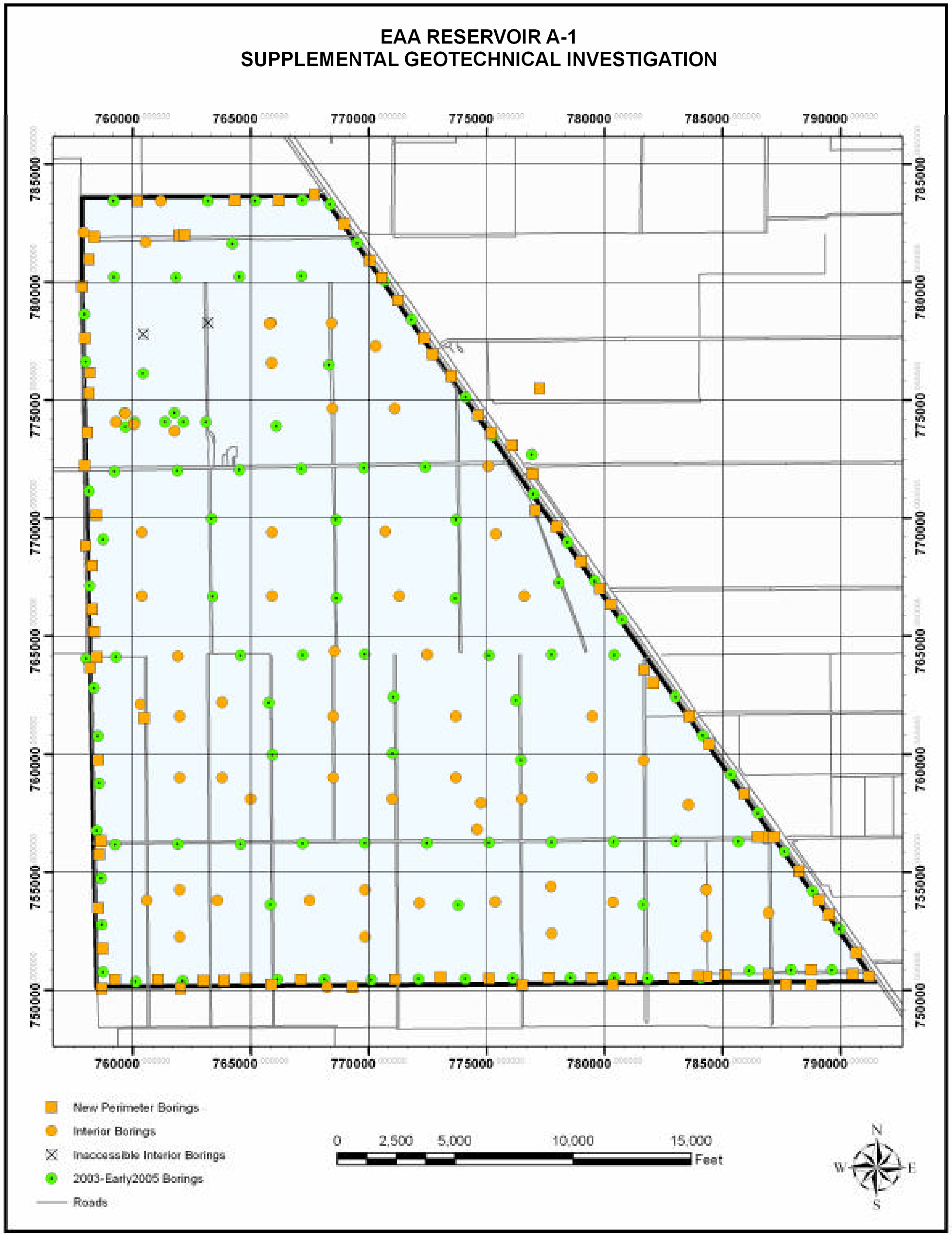
One hundred forty-five geotechnical borings were completed under separate contract for the SFWMD around the planned EAA Reservoir A-1 in 2003 and early 2004. The borings were completed with rotary-wash drilling and standard penetration testing, and were generally between 50.5 and 52 feet deep. Fifteen of the borings had depths ranging from 80 to 91 feet.

Twenty geotechnical borings were completed at the EAA Reservoir A-1 Test Cells site in December of 2004, 10 at the site Borrow Area and five at each Test Cell. The borings were all drilled to a depth of 50 feet, primarily by rotary-wash drilling using a heavy drilling mud to support the borehole sidewalls. The near surface limestone (caprock) was cored in each of the borings, and a deeper, thinner limestone was cored at about 25 feet depth in two of the borings. Soils were tested with standard penetration tests.

Eight additional rotary-wash borings were performed at the Test Cell site during construction of the Test Cells. The caprock in these borings was not cored. Soils were sampled using standard penetration tests.

A supplemental geotechnical investigation was performed at the EAA Reservoir A-1 site concurrent to the preparation of this BODR. Information from that investigation has not been completely processed at the time of this BODR preparation. Figure 7.1-1 shows all boring locations at the EAA Reservoir A-1 Project site. Data from the supplemental investigation will be presented in the Geotechnical Data Report which will be appended to this BODR.

Figure 7.1-1 Boring Locations



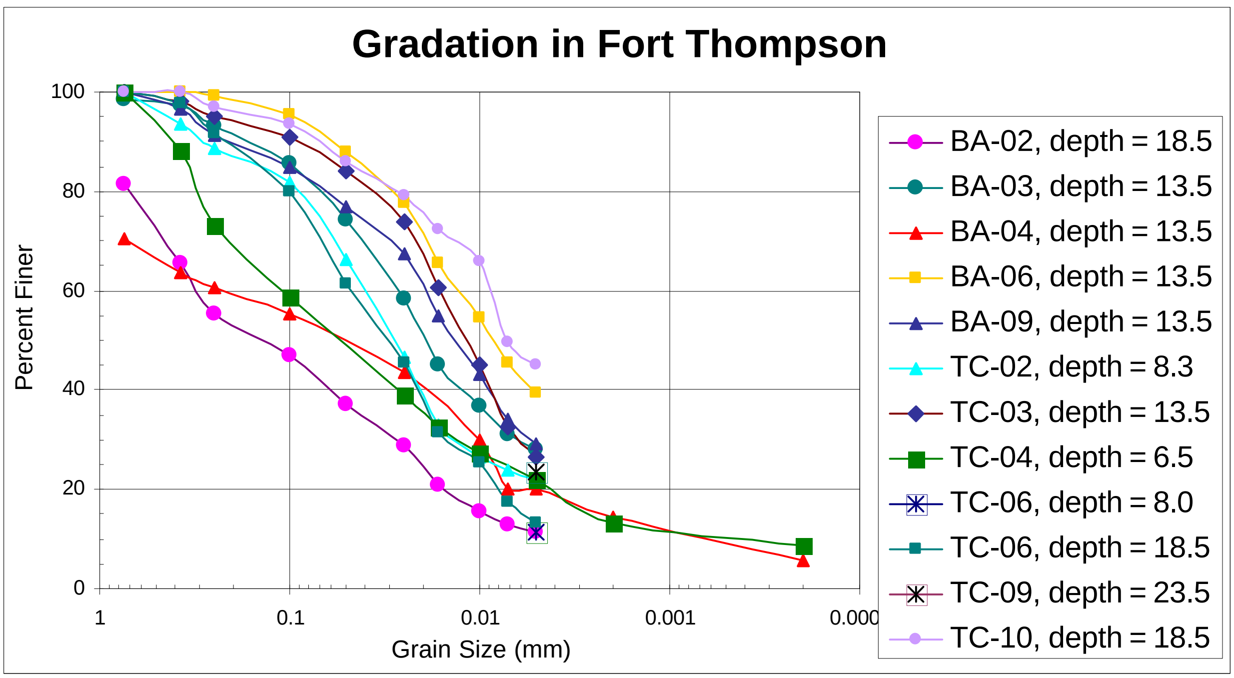
7.2 STRATIGRAPHY

Stratigraphy at the EAA Reservoir A-1 is discussed in Section 2.3.2.

7.3 LABORATORY TEST RESULTS

Twenty samples of the limestone cores from the December 2004 borings performed at the Test Cell site were tested for unconfined compressive strength (ASTM D2938). Rock quality test, including LA Abrasion (ASTM C535) and soundness testing (ASTM D5240) were performed on samples of filter, drain, and riprap bedding produced during construction of the Test Cells. Selected samples of the soils were tested for gradation (ASTM D422), moisture content (ASTM D2216), carbonate content (Florida Test Method Designation FM 5-514), percent passing the No. 200 sieve (ASTM D1140), consolidated undrained triaxial tests, unconsolidated undrained triaxial tests, and flexible wall permeameter tests (ASTM D5084). Laboratory test results associated with the Test Cell Program are presented in Appendix 8-9. A plot of gradation test results from samples obtained in the Fort Thompson Formation is included in Figure 7.3-1. A plot of gradation test results from samples obtained in the Caloosahatchee Formation is included in Figure 7.3-2. The carbonate content of the soil in the Fort Thompson Formation is higher than the carbonate content of the soil in the Caloosahatchee Formation as indicated in Figure 7.3-3.

Figure 7.3-1 Test Results from Fort Thompson Formation



Laboratory testing will be performed on samples of soil and rock cores from the borings being performed in the supplemental geotechnical investigation (Work Order 9). The laboratory tests on soil include grain size analysis (gradation), carbonate content, minus No. 200 sieve tests, and corrosivity tests. Rock quality testing will include specific gravity and absorption, Los Angeles abrasion, soundness, and unconfined compression tests.

Figure 7.3-2 Test Results from Caloosahatchee Formation

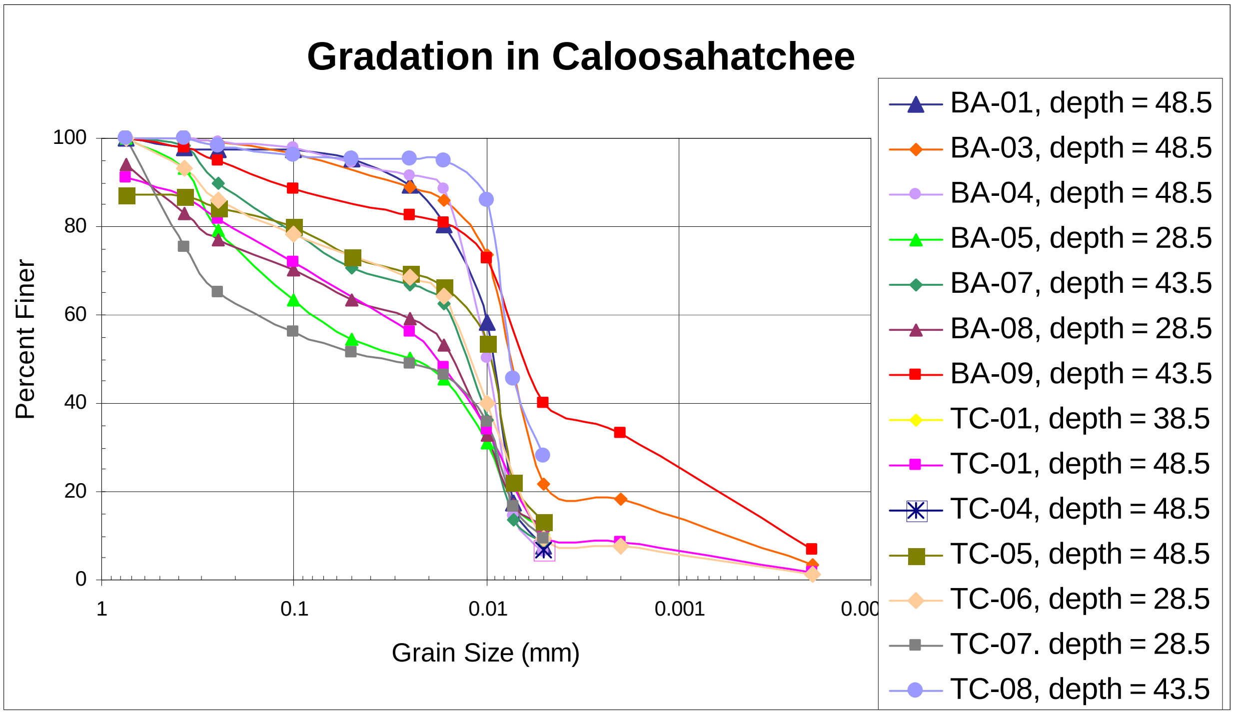
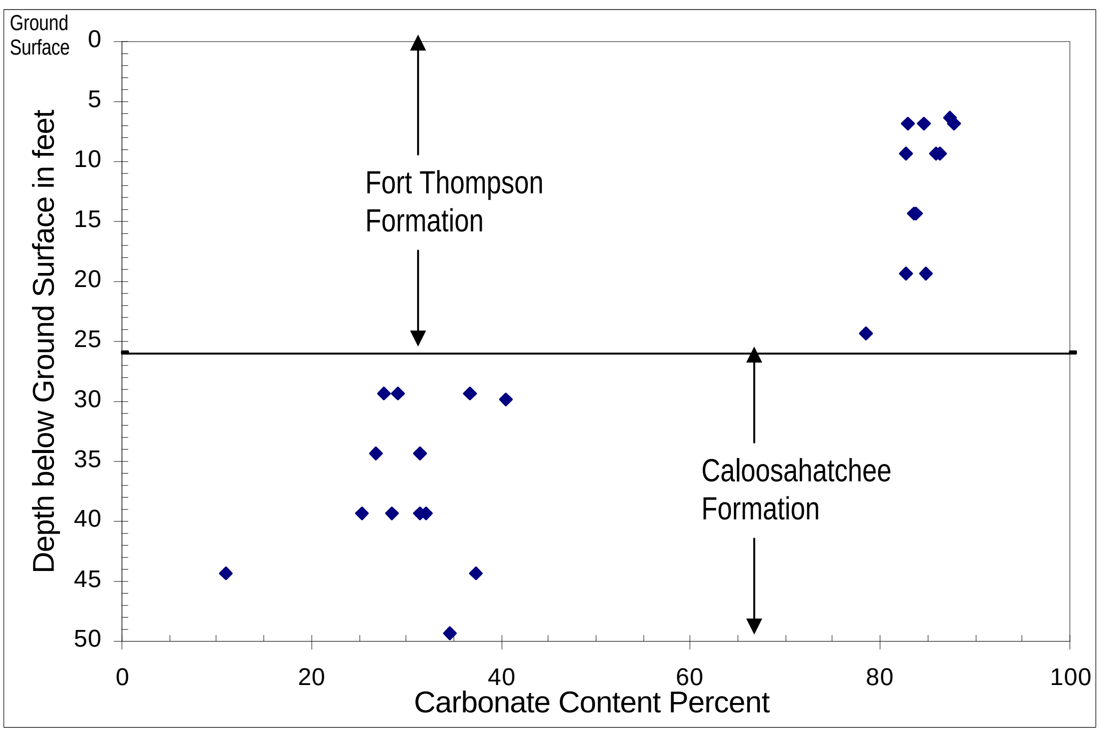


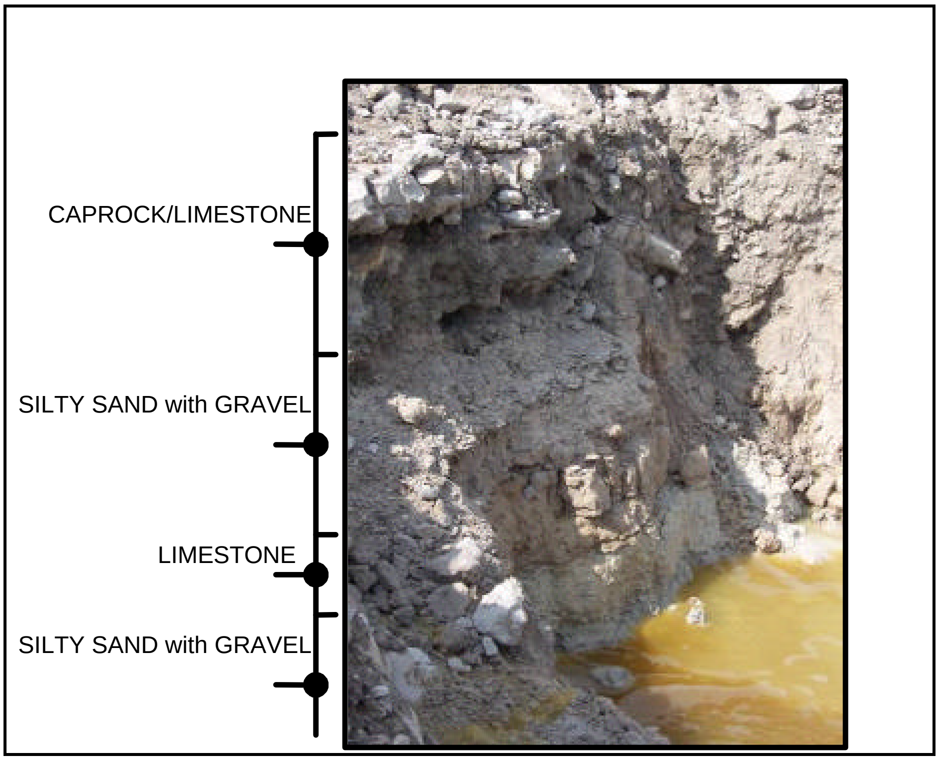
Figure 7.3-3 Carbonate Content of the Soil at the Test Cell Site



7.4 BORROW

The borrow material for the EAA Reservoir A-1 will consist of the caprock and silty sands of the Fort Thompson Formation. The main source of borrow will be the seepage collection canals with additional material being provided by borrow areas within the EAA Reservoir A-1. The caprock and silty sand of the Fort Thompson Formation are visible in Figure 7.4-1. The caprock would provide the source for rockfill, drainage aggregates, gravel surfacing and aggregates for roller compacted concrete for the project. The silty sands of the Fort Thompson Formation would be the source of random fill for construction of an earth/rockfill embankment. Excavation into the Caloosahatchee Formation is not anticipated for a source of materials for the EAA Reservoir A-1 embankment construction.

Figure 7.4-1 Caprock and Silty Sand of the Fort Thompson Formation



7.5 GROUNDWATER

The surficial aquifer system consists of surficial peat/muck and organic soils underlain by the Fort Thompson Formation of late Pleistocene age, the Caloosahatchee Formation of early Pleistocene age, and the upper portions of the Tamiami Formation of Pliocene age. The confining unit at the base of the surficial aquifer system consists of the lower portions of the Tamiami Formation and the upper portions of the Hawthorn Group (Miocene age).

The limestone layers in the Fort Thompson Formation appeared to be the primary source of groundwater seepage into the site excavations made during construction of the Test Cells. Water could be seen streaming from the bottom of each of the three limestone layers in the dewatered excavations. The limestone layers contain fractures. The caprock contains interconnecting solution channels especially near the top, and single channels up to several inches in diameter that penetrate the full thickness of the layer. The solution channels in the caprock locally contain soil including the peat and marl. Furthermore, the unconformity between the caprock and silty carbonate sand near the top of the Fort Thompson Formation appears to act as a conduit for increased horizontal groundwater flow. All of these observed local variations in hydraulic conductivity have implications for the design and specification of the seepage control measures for the EAA Reservoir A-1 embankment, as discussed in Section 8.

7.6 EXCAVATIONS

The existing peat shall be stripped from the caprock surface during preparation of the EAA Reservoir A-1 embankment and seepage collection canals. During Test Cell construction, the majority of the peat stripping was performed with agricultural scrapers and tractors. Stripping started with disking of the areas to promote drying. The surficial peat and marl was stripped from the entire footprint of each Test Cell including the seepage collection canals and the bench between the embankment and seepage collection canals. The areas with deeper or wet materials were completed with dozers pushing the soil into piles that were loaded to dump trucks with excavators. The stripped materials were transported to the perimeter of the active construction area and placed in berms to exclude water discharged from the seepage collection canal dewatering operations that was pumped to the areas outside of the embankments.

Blasting is the typical method for breaking up the caprock for excavations in the EAA Reservoir A-1 area. The seepage collection canals at the Test Cell site were 20 feet deep, 20 feet wide at the bottom and had 2H:1V side slopes. The canals were drilled and shot, generally with a pattern of three blast holes across the canal width. The depth of the shot holes drilled into the silty sand was shortened to minimize mixing of the shot materials. The initial pattern used in Test Cell No. 2 consisted of a 20-foot central hole flanked by two 10-foot holes. This was modified to a 15-foot central hole in the last shot in Test Cell No. 2 and the first hole in Test Cell No. 1. The final two shots in Test Cell No. 1 used only 10-foot blast holes.

The excavation of the seepage collection canal and borrow areas will be performed in a sequence to effectively separate material types. First the caprock should be removed and transported to the embankment, the rock processing plant, or other stockpiles. During Test Cell construction, the contractor chose to dewater the seepage collection canal area prior to excavation of the silty sand. The dewatering operation involved excavating a narrow trench to the planned bottom of the seepage collection canal and installing pumps in sumps below the invert level of the canals. The method of dewatering used for the Test Cell construction is the typical dewatering method used for excavations in the EAA region. Details on the construction dewatering methods used at the Test Cell site are contained in Appendix 8-9.

After the caprock is removed, the underlying silty sand can be stockpiled directly along side the seepage collection canals to promote drainage of the excess moisture from the material. During Test Cell construction hydraulic excavators were used to remove the silty sand from the seepage canals.

7.7 CRUSHING/MATERIAL PROCESSING

Creation of aggregates from the caprock will require crushing, screening, and washing. The caprock contains solution cavities and fractures filled with peat and marl. Because of the high groundwater table, the peat and marl remains moist and sticks to the caprock when excavated. Also, it is not possible to completely remove the muck/peat from the caprock surface during the stripping operation. A roller grizzly is typically used to improve the effective cleaning of processed rock in Florida.

The wet silty sands excavated from the seepage collection canals during Test Cell construction were stockpiled adjacent to the canal excavations. It was observed that even after several weeks in the stockpile the moisture in the soil was above the desired handling and placement moisture content. It was observed that the soil on the surface of the stockpile dried and formed a hard crust. The crust appears to seal the moisture as well as shed rainwater. The wet silty sand placed on the Test Cell embankments also formed a surface crust from one to two inches thick after one to two days of exposure to the sun. The fill placed in an earth/rockfill embankment may require discing, harrowing, or turning with motor graders to reduce the moisture content to the specified limits.

7.8 DESIGN PARAMETERS

Design parameters for the EAA Reservoir A-1 construction are included in Section 8.

SECTION 8
EMBANKMENT/DAM DESIGN

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TABLES

FIGURES

8. EMBANKMENT/DAM DESIGN

8.1 GENERAL

This section summarizes the evaluation and selection of the embankment section proposed for development of the EAA Reservoir A-1. The outline design considered industry standard design criteria as well as various draft Design Criteria Memoranda (DCM) issued jointly by SFWMD, USACE, and FDEP as listed below in Section 8.3.

The studies used factual information obtained from the Test Cell Program, and data obtained from previous soil boring programs. The information from these programs helped to understand the behavior of the insitu materials when excavated and placed on a large scale under accelerated construction conditions and to assess suitability of available borrow resources.

Subsurface conditions dictated the recommended solution. Stability, seepage control, and erosion protection were considered, as well as potential foundation and embankment settlement.

This Section is in two parts:

- First, in Section 8.2 the design concepts and requirements of both concrete gravity and earthfill embankments are discussed, with the advantages and disadvantages of each presented; specific design considerations are addressed.
- Then, the design criteria and parameters used to develop five potential embankment cross sections (two concrete, three earthen) are presented. The cross sections have been developed in sufficient detail, with cost estimates, to allow the selection of the preferred alternative. The cost estimates include the provision of various configurations for upstream wave protection.

The cost estimates presented in Sections 8.12 and 8.13 are for alternative screening purposes. The opinion of probable cost for the full EAA Reservoir A-1 development (including pump stations, gate control structures, and access facilities) is presented in Section 23.

8.2 CONCEPTUAL EMBANKMENT/DAM TYPES

8.2.1 General

As introduced in Section 8.1, two fundamental types of embankment/dam were considered for this site:

- Concrete gravity type dam using roller compacted concrete (RCC)
- Embankment (including concrete faced rockfill)

Each of these types was considered in detail during the preparation of this BODR. A number of alternative arrangements for each type were considered and an opinion of probable cost prepared to evaluate the cost effective aspects of each alternative. The advantages, disadvantages, and risks of each section were considered. The detailed evaluations and findings are summarized in Section 8.11 and presented in Appendix 8-1.

The decision of embankment type must consider initial and long-term stability, seepage control, foundation conditions, and probable costs with appropriate allowances for risks, uncertainties and the cost of mitigation measures. The construction sequence and requirements for each alternative has been considered in detail, the opinion of probable costs is presented in

Section 8.13. The initial, most favorable concrete dam and embankment sections are presented below.

Also presented below is an alternative embankment arrangement that has been evaluated for the southern and southwest portions of the perimeter embankment which incorporates the existing levee and seepage collection canal on the boundary with STA-3/4 and the Holey Land into the EAA Reservoir A-1 embankment. There is no concrete gravity dam alternative to this section in that a RCC dam would have to be constructed on the EAA Reservoir A-1 side of the existing seepage canal for these southern and southwest portions of the perimeter embankment, and no embankment volume reduction would be realized. The EAA Reservoir A-1 volume would be somewhat reduced with the concrete gravity dam option in this location.

8.2.2 Concrete Gravity Dam - RCC Dam

8.2.2.1 General Description

A RCC gravity dam section depicted in the current Tentatively Selected Plan prepared by the USACE Jacksonville District is composed of three stepped RCC sections with a vertical face on the interior of the dam (Figure 8.11-4). An alternative section with potential constructability advantages, but requiring a significant larger volume of RCC, is presented in Appendix 8-1, Figure EMB TM-5. The aggregate for RCC would be obtained from on-site caprock/limestone materials excavated from the seepage collection canals and borrow areas excavated within the EAA Reservoir A-1 interior.

RCC construction generally has an advantage over other types when access is difficult or limited, and often when the required construction schedule is relatively short. None of these conditions exists at the EAA Reservoir A-1 site. The site is open and flat, the embankment is lengthy (22 miles) and access is basically unlimited; these conditions allow multiple construction spreads to operate uninhibited without limitation to the type of embankment considered.

One major disadvantage to RCC is the volume of processed material required for construction. Production of RCC requires a relatively large scale batching operation (crushers, pug mills, aggregate bins, chillers, water supply and cement/fly ash silos) with more limiting output. This batching operation is generally confined to a single large central facility. Due to the extended perimeter length, batching at this site would likely be performed at multiple batching sites. A cost comparison, as described below, is required to evaluate the impact of the higher unit cost.

Many normally recognized advantages for using RCC are basically lost, or do not exist, at this site. Advantages and disadvantages are discussed in more detail in the following subsections.

8.2.2.2 Advantages

The advantages of this design include:

- Occupies smaller footprint than an earth filled embankment, so EAA Reservoir A-1 storage can be maximized
- Provides its own slope protection
- Reduced volume of material to be placed
- Minor overtopping may be acceptable without damage
- Downstream slope maintenance generally not required

8.2.2.3 Disadvantages

The disadvantages of an RCC dam design at this site include:

- Without a wave break, larger freeboard is required because the vertical face results in a higher wave run-up
- Transverse cracks are predicted to occur due to existing foundation conditions
- Predicting the magnitude of cracking and control of crack location with certainty is difficult due to the variable foundation conditions
- Seepage is expected to occur through cracks without waterstops due to differential movement
- Special foundation treatment measures are required where caprock does not exist or where the caprock is determined to be thin (less than three feet thick – USACE criteria)
- Settlement of the cutoff wall must be compatible with the rigidity of the RCC structure
- Foundation preparation by air/water blasting will be required
- Grout treatment of the foundation contact area will be required
- Requires a large volume of cement to be imported for construction, which increases the risk of supply shortages
- Requires larger volume of processed aggregates than an embankment
- Temperature control of the RCC components and mixture will be required during high temperatures
- Utilizes less volume of the silty sand material resulting from excavation for the seepage collection canal, therefore spoil volume is increased and removal of excess spoil results in additional cost
- Cannot be combined with existing embankment adjacent to STA-3/4 and Holey Land Tract, therefore reduces EAA Reservoir A-1 volume
- Concrete structure does not fit aesthetically with the surroundings
- Vertical upstream face creates a safety hazard due to potential falls; a means for escape for anyone falling into the EAA Reservoir A-1 would be required
- Requires additional access arrangements to allow for safe and convenient recreational use

8.2.3 Embankment Design

8.2.3.1 General Description

An embankment concept has been developed to utilize materials from the required seepage collection canal excavations and available on-site borrow resources, and to minimize sorting and processing of the excavated materials for embankment construction (Figure 8.11-1). The rockfill zone material will be produced from the caprock providing structural stability to the upstream slope. Between the rockfill and random fill zones, a transition material is provided to protect against migration of fines from the random fill material zone (rock size particles to minus No. 200 sieve) into the rockfill by action of gravity during drawdown and water level change that the embankment will experience.

A vertical filter (chimney drain) is provided for internal piping control and drainage, and to control the phreatic line in the downstream random fill slope. The filter gradation and its width will be designed to ensure sufficient capacity to eliminate the need for a two material filter/drain system.

A horizontal blanket filter extends over the caprock to relieve seepage pressures and control loss of infilled fine-grained material from the caprock and upper silty sand foundation. A gravel toe drain is provided at the downstream toe of the embankment. The location of seepage from the drain will be concentrated at low points in the caprock topography.

The downstream 3H:1V slope of the embankment will include a layer of organic material from the stripping and the embankment will be seeded to allow for maintenance in accordance with the SFWMD Standard Design criteria.

8.2.3.2 Advantages

The advantages of this design include:

- All materials resulting from the required excavation of the seepage collection canal can be used in the embankment
- The material suitability of available on-site materials and construction process has been confirmed in the Test Cell Program
- Apart from the cement for the slope protection and bentonite for the cutoff wall, all materials for embankment construction can be produced on site
- Processing of excavated material for embankment construction can be limited to crushing, screening, washing, and blending for drain materials, riprap bedding and aggregates for RCC slope protection
- Stripping to the caprock surface and cleaning using a rotating power broom is adequate foundation preparation beneath the embankment except in local disturbed areas
- The seeded downstream slope blends in with the general surroundings to provide aesthetic value
- The embankment has the flexibility to accommodate settlement in the foundation
- In the extremely unlikely event of earthquake loading, earthfill embankments are very resilient and are flexible enough to absorb some foundation movement
- The embankment deformation and slurry cutoff wall settlement are compatible, thereby maintaining the interface connection
- SFWMD personnel are familiar with the maintenance requirements for embankments

8.2.3.3 Disadvantages

The disadvantages of the earth filled embankment design include:

- The material from the Fort Thompson Formation, which will be used as random fill, is very wet and difficult to work when taken directly from the excavation. However, experience from the Test Cell Program and SFWMD's levee construction has demonstrated that the materials can be used successfully when allowed to dry or drain
- Armored slope protection, such as RCC, will be required for wave protection
- Requires more frequent surveillance of upstream slope protection
- Requires more instrumentation for surveillance/monitoring than a RCC gravity dam
- The potential for internal erosion and piping is higher than for a RCC gravity dam
- Overtopping is not allowed

8.2.4 Embankment Section along STA-3/4 Supply Canal

8.2.4.1 General Description

Along the south side of the EAA Reservoir A-1 adjacent to STA-3/4 and the Holey Land Tract, an earthfill embankment offers the option of combining new and existing structures. The embankment section depicted in Figure 8.11-7 shows how the existing seepage canal and perimeter levee for the STA-3/4 can be utilized to minimize the volume of new embankment construction. A similar concept has not been considered for the RCC concept due to anticipated foundation conditions, potential settlement, and seepage control considerations.

8.2.4.2 Advantages

For an embankment, the advantages of this alternative include:

- Less stripping of peat required
- Caprock excavation is not necessary for construction of key trench or a cutoff wall installation. Existing seepage collection canal which has been excavated through the caprock and upper limestone can be used for key trench type of foundation cutoff
- Embankment volume is reduced by incorporating existing perimeter levee
- Crest of existing perimeter levee can be used as access road
- Maximizes EAA Reservoir A-1 volume potential along STA-3/4 canal and north-south segment paralleling the Holey Land area
- Seepage from EAA Reservoir A-1 into STA-3/4 will keep the STA-3/4 hydrated

8.2.4.3 Disadvantages

The disadvantages of this method include:

- Seepage into STA-3/4 Supply Canal and feeder canal would be partially uncontrolled
- Dewatering in existing interior seepage canal is required during fill placement

- Dewatering of the existing Supply Canal would be limited to periods of low to minimal water level in the EAA Reservoir A-1 to reduce seepage pressure into the dewatered Supply Canal

8.3 DESIGN CRITERIA

8.3.1 Sources

United States Army Corps of Engineers (USACE) Design Manuals:

- Engineering Manual, EM 1110-2-1902, Engineering and Design: Slope Stability, 31 October 2003
- Engineering Manual, EM 1110-2-2006, Roller-Compacted Concrete, 15 January 2000
- Engineering Manual, EM 1110-2-2200, Gravity Dam Design, 30 June 1995
- Engineering Manual, EM 1110-2-2300, Earth and Rock-Fill Dams, General Design and Construction Considerations, 30 July 2004

Acceler8 Design Criteria Team, Design Criteria Memoranda:

- 'Hazard Potential Classification,' DCM-1, 19 August 2005
- 'Minimum Dimensions of Dams and Embankments,' DCM-4, 9 August 2005
- 'Geotechnical Seismic Evaluation of CERP Dam Foundations,' DCM-6, 16 May 2005

8.3.2 Embankment Slope Stability Factors of Safety (FoS)

The minimum required factors of safety for each embankment design case are as follows:

Design case	Factor of safety
End of construction	1.3
Steady seepage at normal pool level	1.5
Steady seepage with surcharge pool	1.3
Steady seepage with earthquake loading	1.1
Rapid drawdown from normal pool	1.3
Rapid drawdown from surcharge pool	1.1

8.3.3 Gravity Section Stability Criteria

Gravity embankment stability criteria are given in Table 8.2-1.

Table 8.3-1 Gravity Embankment Stability Criteria

Load Condition	Location of Resultant	Min FoS against Sliding	Foundation Bearing Pressure	Concrete Stress	
				Compression	Tension
Usual	Middle 1/3	2.0	= allowable	0.3 fc’	0
Unusual	Middle 1/2	1.7	= allowable	0.5 fc’	$0.6\ fc'^{2/3}$
Extreme	Within base	1.3	= 1.33 x allowable	0.9 fc’	$0.9\ fc'^{2/3}$

8.3.4 Water Levels

The Maximum Hazard classification of this embankment requires that the EAA Reservoir A-1 be sized to store the PMP as described in Section 5.2. A PMP of about 4.5 feet was used as the basis for the work presented here. The total embankment height will depend on the normal water level plus the freeboard requirements. Freeboard allowance is determined from the effects of wind and rainfall and other considerations as described in Section 5.5.

Modeling was conducted to determine the effectiveness of a perimeter bench on wave run-up thereby reducing the required freeboard. A perimeter bench would be effective in reducing wave run-up on the embankment. For modeling purposes, the bench was set at a depth of 19.05 feet. For a 25-foot wide bench the maximum water level ranges from 21.1 to 22.0 feet for the zoned embankment and from 20.8 to 21.9 feet for the RCC dam. For a 15-foot wide bench the maximum water level ranges from 21.3 to 22.4 feet for the zoned embankment and from 21.8 to 22.4 feet for the RCC dam. A submerged bench would also be effective in breaking the incident wave and reducing wave run-up.

The embankment height should be set higher than the Maximum Water Level (MWL) to prevent overtopping to account for the irregular nature of waves. Without a perimeter bench, the embankment height would need to be about 26 feet above the EAA Reservoir A-1 bottom for a zoned embankment and about 28 feet above the EAA Reservoir A-1 bottom for an RCC dam to prevent overtopping. The overtopping analysis indicates that the embankment height can be significantly reduced with the addition of a perimeter bench. With a perimeter bench, an embankment height of 22 feet above the EAA Reservoir A-1 bottom would prevent overtopping for both types of embankments. An evaluation of wave erosion protection measures is presented in Section 8.12.

The RCC section height can be achieved by extending a parapet wall above the base structure. These elevations may change as the design develops. Embankment features to reduce the wave heights and potentially lower the crest elevations have been considered and are discussed below.

8.3.5 Seismic Loading

For the pseudo-static stability analysis of the embankment, a 0.05 gravity horizontal acceleration coefficient and a 0.025 gravity vertical acceleration coefficient were applied simultaneously. A pseudo-static method of stability analysis was used because the EAA Reservoir A-1 site is in an area of low seismicity. The values chosen for the analysis are conservative given the low seismicity of the site.

Design Criteria Memorandum 6 (DCM-6) requires an evaluation of the liquefaction potential of the embankment foundations. The method of evaluation is based on assessment of continuous Standard Penetration Tests (SPT) in boreholes and comparison with standard design charts. This evaluation will be made when the data is available from ongoing geotechnical investigations.

8.3.6 Embankment Cross Section Configuration

DCM-4 sets out the following criteria for embankments. The criteria were developed in coordination with the USACE. No equivalent criteria were available for concrete gravity dam sections such as the RCC sections considered.

- DCM-4 requires a minimum crest width for CERP impoundments of 12 feet for an embankment with 3H:1V side slopes, with a note that the final design crest width will be based on engineering analysis for seepage and slope stability requirements, analysis of the cost effectiveness for alternative embankment sections, and the constructability of the section. Black & Veatch recommend a 14 foot crest width because 12 foot width is too restrictive during construction for safe and efficient material handling, and to accommodate large heavy maintenance equipment in the long term.
- Based on experience, slopes will be 3H:1V or flatter, for the design of embankments to provide a suitable area for O&M activities, specifically mowing of the exterior slope. The final embankment slopes will be based on technical consideration of seepage, slope stability, and protection required to resist erosion from wave action or surface runoff.

8.3.7 Perimeter Access Requirements

DCM-4 establishes the following criteria for perimeter access that impacts embankment layout and embankment construction costs:

- A Perimeter Access/Maintenance/Inspection Corridor (Perimeter Corridor) at least 50 feet wide is to be provided around the exterior perimeter toe of the embankment. The Perimeter Corridor is to include an 18-foot wide all-weather (gravel) surface. An interior corridor (Interior Corridor) at least 50 feet wide is to be provided around the interior to allow access when the EAA Reservoir A-1 is drained.
- The Exterior Perimeter is to consist of a clear, even graded area for safety inspection, but can also be used for maintenance and access to the embankment and canals, and utilities on a select basis.

Access requirements for execution of the required seepage collection canal excavation and construction access for future construction of EAA Reservoir A-2 have been considered.

8.4 *EMBANKMENT/DAM MATERIALS*

8.4.1 General

The economic feasibility of the EAA Reservoir A-1 embankment is dependent on effective utilization of the available on-site materials during construction to the greatest extent possible. The development of the Test Cells and perimeter seepage collection canal allowed an evaluation of the suitability of on-site materials for embankment construction and erosion protection.

The proposed seepage collection canal on the exterior of the EAA Reservoir A-1 embankment is a source of construction material. Additional materials will be obtained from borrow areas excavated in the EAA Reservoir A-1 interior.

It is not possible to use the insitu materials for all the elements of the structure and some materials must be imported; this would be particularly true for a concrete dam or where a geomembrane is used as the watertight barrier. Table 8.4-1 shows the availability of construction materials on site; the evaluation of the embankment alternatives requiring a range of materials is described in Section 8.11.

Table 8.4-1 Availability of construction materials

Embankment element	Material	Availability
Watertight barrier	Insitu soils (Fort Thompson)	On site
	Bentonite (for a cutoff wall)	Imported
	Cement (for a concrete dam)	Imported
	Geomembrane	Imported
Shoulder support	Insitu soils (Caprock and Fort Thompson)	On site
	Concrete (for a concrete dam)	Imported (cement)
Internal drain	Caprock (crushed)	On site
Foundation drain	Caprock (crushed)	On site
Road stone	Caprock (crushed)	On site

8.4.2 Subsurface Profile

The insitu materials at the EAA Reservoir A-1 site have been investigated by a series of borings performed in 2003 and early 2004, and borings completed for the Test Cell Program in December 2004 and early 2005. These borings were performed using rotary wash drilling and core drilling techniques. SPTs were made in the borings and samples retrieved for laboratory testing. Material classification, gradation, and rock core strength tests were performed on appropriate representative samples.

The generalized subsurface profile defined from the investigation information and used in the embankment design is as follows:

- Surficial peat and marl: The peat (also referred to as “muck”) is a black, highly organic, fine grained soil with a variable thickness of one to two feet. In generally isolated areas, the muck is underlain by several inches to two feet of calcareous clay (locally called “marl”).

- Caprock/upper limestone (Fort Thompson Formation): Hard limestone layer (generally referred to as “caprock”) varying in thickness from zero to about 13 feet across the EAA Reservoir A-1 site. Thickness in the Test Cell Program site varied from about two to five feet. The upper one to three feet of this layer is solution riddled and commonly sandy or shelly.

- Silty carbonate sand with limestone layers (Fort Thompson Formation): Silty carbonate sand containing shell fragments, tends to be angular and platy, extending to about 23 to 35 feet depth bgs across the EAA Reservoir A-1 site; average calcium carbonate content is 84.2 percent; average 22 percent passing the No. 200 sieve.

- Sand with sparse limestone layers and intervals of hard drilling (Caloosahatchee Formation): Shelly, fine-grained, subrounded, quartz sand mixed with shelly carbonate sand starting at a depth of about 100 feet bgs. Proportions of calcium carbonate to quartz vary greatly; few short intervals of hard drilling less than one foot encountered in some borings; average calcium carbonate content of 40.1 percent and average of 12.1 percent passing the No. 200 sieve.

Detailed descriptions of the insitu materials encountered in the Test Cell Program are included in Appendix 8-9. Boring logs and associated laboratory testing are included in Appendix 8-17. A program of additional investigation and testing for the EAA Reservoir A-1 Project’s design is currently underway.

Insitu materials are appropriate for use as:

- Water tight barrier
- Rockfill shoulder support
- Internal drainage
- Foundation drainage
- Road surface

Insitu materials below a depth of about 30 feet are not anticipated to impact the material selection and zoning for design of the perimeter embankment at the EAA Reservoir A-1 site.

8.4.3 Watertight Barrier Materials

8.4.3.1 General

The primary purpose of an embankment is to retain water, thus the watertight element is the most important part of the structure. A variety of materials have been used for this component in embankments constructed throughout the world. The four alternatives evaluated for this site included:

- Silty sand from the Fort Thompson Formation for the watertight barrier in a zoned embankment

- Geomembrane encapsulated in a silty sand zone of an embankment

- Reinforced concrete face for a rockfill embankment section
- Concrete for the two RCC dam alternatives

8.4.3.2 Embankment Material

Test Cell construction demonstrated the feasibility of constructing a zoned earth/rockfill embankment with a water detention zone (select fill) derived from the fine silty sand materials excavated from the Fort Thompson Formation. In the internal water detention zone of the embankment, rock particles greater than six inches can be raked out of the available silty sands of the required excavations or borrow areas using standard earthmoving equipment to achieve an adequately low permeability zone.

8.4.3.3 Roller Compacted Concrete (RCC)

RCC is concrete placed in large quantities at a low moisture content which allows the economic use of earthmoving type equipment. RCC serves as the watertight element in a RCC gravity dam. Often a grout enriched upstream RCC zone, or conventional concrete facing, is incorporated into the RCC mass section to provide a more durable and watertight upstream face. Due to the low head design for this project, the conventional concrete concept for the upstream face has not been considered.

RCC is not considered to be suitable as a watertight membrane on the upstream face of an earthen embankment because of the cracks that generally form from both shrinkage of the concrete and settlement anticipated in the fill and due to variable foundation conditions.

8.4.4 Rockfill

The outer rockfill shoulders of a zoned embankment provide the support to ensure stability of the watertight internal element. Outer rockfill sections are normally included when the appropriate materials are available. At the EAA Reservoir A-1 site, rockfill can be obtained from caprock/limestone blasted from the seepage collection canal excavation and internal borrow excavations. Rockfill can also be used to form the upstream wave break bench in the alternative evaluations.

8.4.5 Internal Drainage

8.4.5.1 Crushed Aggregates

The vertical chimney and horizontal blanket drains in the embankment serve to control pore pressures and the location of the phreatic line within the embankment. Horizontal drains also intercept and collect seepage through the embankment foundation. The purpose of the chimney drain is to intercept seepage through more permeable horizontal layers inherent in the embankment construction to prevent saturation of the downstream slope and to prevent seepage exiting the downstream slope above the toe. Flow intercepted through the chimney drain is conveyed to the horizontal drain. Coarse and fine aggregates for use in the internal drainage system can be crushed and screened from the caprock/limestone layer. The grading of the material and thickness of the drains will be refined during the design phase; they must meet filter criteria with the surrounding materials and have sufficient hydraulic capacity to carry the expected flows.

8.4.5.2 Pressure Relief Drains

Internal drainage in a concrete gravity dam would normally be provided to relieve water pressures which can build up on concrete lift interfaces and joints. Relief drains are generally constructed in the form of cast-in-place pipes or drill holes.

8.4.6 Foundation Drainage

The crushed aggregate horizontal blanket drain in an embankment functions to intercept seepage that emerges from the foundation and to provide the controlled release of excess pressure along the foundation contact. For an embankment founded on the caprock foundation at this site, the blanket drain serves to:

- Relieve uplift pressure caused by seepage
- Permit discharge of seepage water from the foundation
- Prevent piping of fines from the embankment and foundation
- Convey the seepage to the downstream (or exterior) toe of embankment slope.

Foundation drainage may be required for a concrete gravity dam to enhance stability by reducing uplift pressures. Such drainage is often provided by using drains drilled in the foundation. Proper filtering is required to prevent loss of fines from the foundation materials.

8.5 STABILITY

8.5.1 General

For the evaluation of dam alternatives, stability analyses were performed for an embankment and RCC dam configuration to define critical stability issues to be considered. The stability analyses for an embankment section were performed using presumptive strength values in that detailed geotechnical investigations and associated laboratory testing will not be finalized prior to submittal of the BODR. The presumptive strength values used in the analyses were chosen based on preliminary testing from the Test Cell Program and engineering judgment through experience with testing of silty sand materials on other projects.

The stability analyses were performed for two embankment sections of material zoning developed from observations and experience gained during the Test Cell Construction and Monitoring Program. Additional stability analyses of the design alternative(s) advanced to 30 percent design will be performed using strength test results available from the ongoing laboratory testing program. The strength parameters for the RCC dam section were obtained through information exchanges and discussions with the USACE.

8.5.2 Material Parameters

The parameters used in the stability analysis are listed in Table 8.5-1. These parameters are considered to be conservative for use at the BODR phase of the project.

The effect of the RCC slope protection and the drain material were not considered in the stability analysis. The RCC layer on the upstream slope would enhance slope stability by adding additional mass and strength, therefore excluding this layer is conservative. The filter layers are narrow and of at least equal strength to the adjacent embankment zones, therefore excluding the filter is also conservative and simplified the modeling effort at this stage.

8.5.3 Embankment Slope Stability

The slope stability analyses were performed using the computer program SLOPE/W, version 5.17, by GEOSLOPE International Ltd. as described in the Embankment Technical Memorandum (Appendix 8-1). Two embankment sections were analyzed. Model A includes an upstream rockfill section and Model B consists of a random fill/select fill section, both with 3H:1V, upstream and downstream slopes.

The results of the stability analysis are listed in Table 8.5-2. Satisfactory factors of safety were achieved for all cases analyzed. The output from stability analyses are contained in Appendix 8-2 resulted in a factor of safety larger than USACE acceptance criteria.

Embankment stability will be increased if rockfill sections are considered for the outer slopes. Stability analyses will be reviewed and performed as required to consider the section modifications developed for final design.

8.5.4 Concrete Gravity Dam - RCC

Two alternative RCC sections are included in the embankment evaluations described in Section 8.11. The section presented in Figure 8.11-4 was developed by the USACE. Stability analyses were performed by the USACE on their RCC dam section. The documentation was reviewed and considered when preparing this report.

Stability analyses performed for a second alternative RCC section developed by Black & Veatch are included in Appendix 8-4. The RCC section was evaluated in accordance with EM 1110-2-2006, Roller Compacted Concrete and EM 1110-2-2200, Gravity Dam Design. All stability criteria were met as shown in Table 8.5-3. This RCC section provides a significantly wider base due to the “small step” configuration with the set crest width of 12 feet. However, the RCC volume is also significantly increased. A sketch of the section is provided on Figure EMB TM-5, Appendix 8-1.

Table 8.5-1 Stability Analysis Parameters

Material Type	**?$_{T}$ (pcf[1])**	**?$_{sat}$ (pcf)**	**c’ (psf[2])**	**f ’ (deg[3])**	**c$_{T}$ (psf)**	**f $_{T}$ (deg)**	**Remarks**
Random Fill	115	122	0	35	0	35	Material from required excavations. Fort Thompson Formation
Select Fill (6 inch maximum)	115	122	0	33	0	33	Random fill with material larger than 6 inches removed
Transition zone	115	122	0	33	0	33	4 inches minus crushed run material from caprock
Rockfill	135	140	0	40	0	40	Blasted caprock, no processing
Filter	120	125	0	35	0	35	Not modeled. Derived from crushed and processed caprock
RCC	145	150	5,000	35	5,000	35	Not modeled in earthfill embankment. Aggregates derived from caprock crushed and processed on site
Limestone	134	140	2,000	40	2,000	40	Foundation, caprock layer
Silty Sand with Gravel	120	125	100	33	100	33	Foundation. Fort Thompson Formation
Gravelly Sand	120	125	0	35	0	35	Foundation. Caloosahatchee Formation

[1] Pounds per cubic foot
[2] Pounds per square foot
[3] Degree

Table 8.5-2 Results of Stability Analysis

Case	Strength Parameters	Factor of Safety	
		Upstream Slope	Downstream Slope
Stability Model A			
End of Construction	Total	2.58	2.17
Steady Seepage with Normal Pool	Effective	-	1.92
Steady Seepage with Surcharge Pool	Effective	-	1.87
Rapid Drawdown from Normal Pool	Effective	1.41	-
Rapid Drawdown from Surcharge Pool	Effective	1.34	-
Steady Seepage with Earthquake Loading	Effective	-	1.75
Stability Model B			
End of Construction	Total	2.18	2.17
Steady Seepage with Normal Pool	Effective	-	1.92
Steady Seepage with Surcharge Pool	Effective	-	1.87
Rapid Drawdown from Normal Pool	Effective	1.02	-
Rapid Drawdown from Surcharge Pool	Effective	1.98	-
Steady Seepage with Earthquake Loading	Effective	-	1.75

Table 8.5-3 Stability Criteria Results

Embankment Alternative	Analysis Condition	Factor of Safety	
		Overturning	Sliding
Alternative 5	NWL[1]	Resultant falls in middle 1/3	17.69
Alternative 5	NWL plus PMP[2]	Resultant falls within base	9.70
Alternative 5	NWL plus Seismic	Resultant falls within base	17.17

[1] Normal water level

[2] Probable maximum precipitation

8.6 *EROSION PROTECTION*

8.6.1 General

The upstream slope of an earthen embankment must be protected from damage caused by waves; without protection it is possible that the resulting erosion could breach the embankment causing uncontrolled release of water.

A variety of alternative wave protection systems are used in reservoir and coastal engineering schemes including: riprap, concrete slabs, concrete blocks, roller compacted concrete flat plate, roller compacted concrete stair step, bitumen systems, and various shapes of precast concrete blocks. Typically, the lowest cost protection is provided by using on-site materials if they are suitable. Riprap, produced from on-site rock materials, and RCC utilizing aggregates produced on-site are the two materials potentially most appropriate at this site for wave protection.

DCM-2 described the criteria and techniques required for evaluation of wave action on CERP impoundments. Wave height studies are described in Section 5.5 and Appendix 5-17. The simplest design solution would be to armor the front of the embankment with a protective material; alternatively studies have shown potential benefits of providing a bench in front of the embankment at approximately surcharge pool level. The erosion protection alternatives have significant cost differences and thus influence embankment selection. These are discussed in detail in Section 8.12.

8.6.2 Riprap

Riprap is commonly an economical means of slope protection when a source of suitable rock is available. The rock source must be capable of producing pieces of appropriate size to withstand the design wave forces and have suitable durability characteristics.

The caprock was evaluated during the Test Cell construction for its use as riprap. The evaluation consisted of gradation tests performed on four different blasting trials and rock quality testing. A review of the gradations produced (Appendix 8-24) indicates that the caprock will not produce riprap that is sufficiently large for the design wave height for the embankment unless a wave break bench is provided as described in Appendix 5-17 and discussed further in Section 8.12.

8.6.3 Roller Compacted Concrete (RCC)

RCC is considered the most appropriate means of erosion protection for the maximum height wave predicted for the design wind speed, and combined wind speed and precipitation conditions.

The wave break bench concept described in Section 5.5 would allow the use of RCC to be limited to the top of the bench and slope below the bench, with riprap above the bench for the embankment. The RCC would be installed on a 3H:1V slope at a thickness of 18 inches. A control joint designed to accommodate shrinkage and control of irregular crack development (probably some type of lap joint configuration) should be provided at the top of the slope placement. The top of the slope at original grade, currently considered to be (OG) + 16 feet, is 0.5 feet below NWL + PMP (normal water level plus probable maximum precipitation), which indicates that the concrete joint at break of slope would only be subjected to wetting under wave action by infrequent high wind occurrences or extreme rain events.

8.6.4 Precast Mass Concrete Crest Wall

A precast concrete crest wall in the upper portion of the embankment could potentially be combined with some of the above wave protection alternatives to reduce embankment volume. This alternative will be investigated as a further refinement as design progresses. There are potential reasons to avoid using such a wave wall such as maintenance, aesthetics, constructability, long-term stability of the wall, and effective crest width reduction for access on the upstream slope. Ultimately the cost of the embankment volume saved would need to offset the cost of the crest wall by a sufficient margin to compensate for the disadvantages.

8.7 SEEPAGE CONTROL

8.7.1 General

Seepage control has two principal design functions:

- The first function is embankment and foundation stability: pore pressures and hydraulic gradients must be controlled to protect the embankment and foundation from internal erosion (piping) and to ensure stability
- The second function is to mitigate off-site impacts due to increased seepage

This section of the BODR describes the minimum measures required to ensure stability. Seepage computer modeling has been performed to evaluate seepage control requirements and alternatives. The detailed summary of the modeling is presented in Appendices 8-6, 8-8 and 8-10. Off-site impacts are considered separately in Section 9.

8.7.2 Foundation Seepage Control Alternatives

The aims of the foundation seepage control alternatives are to mitigate seepage losses from the EAA Reservoir A-1, to protect the foundation from possible damage by piping and minimize excess uplift pressures to enhance stability. Historically the mode of operation of the low-head water control facilities in the area has been to accept seepage and return it to the canal system by pumping. With higher head in the EAA Reservoir A-1, foundation stability issues are more critical and economic impacts due to pumping will be experienced on a long-term basis.

Seepage in the embankment will be controlled by the water detention zone and carried safely to the downstream toe by the drainage system. In the foundation, the strata will control seepage at the various depths:

- Caprock - highly permeable due to fracturing and solutioning. Seepage controlled by key trench or cutoff wall.
- Fort Thompson - geological context and depositional environment supports the high horizontal permeabilities derived from the Test Cell observation. Cutoff wall effective for seepage control.
- Caloosahatchee - similar geological context and depositional environment to the Fort Thompson stratum. Seepage could be controlled by a deep cutoff or special provisions such as deep pressure relief wells if required.

Several different configurations to mitigate seepage from the EAA Reservoir A-1 and control exit pressures were evaluated: adequately sized key trench, cutoff wall or upstream blanket, and increasing distance between the EAA Reservoir A-1 and seepage collection canal. These options are described further in Appendix 8-6.

8.7.2.1 Key Trench

The minimum width of a key trench would be governed by the hydraulic gradient across the base of the trench. Limiting the gradient reduces the chance of hydraulic fracturing or piping. Additional protection of the fill against piping would be required on the sides of the trench because of potential open areas or voids and channels through the caprock. This protection could be provided by shotcreting the excavation slope or by filter placement.

8.7.2.2 Cutoff Wall

A cutoff wall forces seepage to pass vertically downward through low permeability material before it can escape the projected EAA Reservoir A-1 perimeter. A foundation cutoff can be installed below the water table using the slurry method of trench excavation. The method involves excavating a trench below the groundwater and maintaining trench stability with a dense thixotropic mixture of water and bentonite. Except for very deep trenches, cutoff trenches are typically excavated with hydraulic excavators. The backfill for the wall is typically mixed on the ground surface adjacent to the cutoff trench. The backfill generally consists of a mixture of the excavated trench soils and processed commercial bentonite. A soil-bentonite cutoff wall is the most economical when suitable site soils are available for use as backfill. However, other types of wall backfill options include soil-cement-bentonite, cement-bentonite, and plastic concrete. These options produce a stiffer wall as may be required to ensure compatibility with a concrete dam, but they are more expensive. Placement procedures vary depending on the backfill option selected.

The caprock at the Test Cell site was observed to have solution voids and channels. Many of the vertical solution channels were circular, ranging in diameter from a fraction of an inch to several inches, and extended all the way through the caprock. The boils that occurred at Test Cell 1 were, primarily, associated with these holes. Bedding planes and more porous layers of limestone are also present between hard and dense layers of limestone within the caprock/upper limestone unit. These porous layers contain horizontal solution channels but the continuity of these channels is not known. The solution voids are generally filled with peat and fine-grained soils. There is a danger that piping could occur through the cutoff wall at caprock level if no protective measure is provided.

A shallow cutoff wall through the caprock and extending a minimum depth of eight feet into the underlying silty sand (Fort Thompson Formation) represents the minimum cutoff depth. The purpose of this wall would be to cutoff flow through the caprock layer to control seepage that might otherwise cause piping of the embankment soil and foundation soil. The width of this shallow cutoff wall must be increased to protect against piping along the base of the cutoff.

The Test Cell Program (Test Cell 1) demonstrated the vulnerability of the foundations when no cutoff wall is provided. At Test Cell 1, boils developed on the downstream side of the embankment within two days of filling to half its design depth (six feet). Long-term seepage could develop into uncontrolled piping of the sandy silt/silty sand soils underlying the caprock around the perimeter of the EAA Reservoir A-1. The general conditions around Test Cell 1 have a high potential to develop piping for an embankment with a significant design life. A surveillance and monitoring program is an important aspect of embankment safety. EAA Reservoir A-1 presents a significantly more difficult surveillance challenge than most embankments, which are generally much shorter. Most embankments are routinely inspected closely on foot for signs of deterioration. Such close scrutiny will be hard to maintain but necessary in these circumstances, so conditions that represent a future vulnerability, such as potential boils and aggressive leakage, should be considered in design as much as possible.

8.7.3 Seepage Model Parameters

The subsurface stratigraphy developed from the borings performed for the Test Cell Program was used in the seepage computer modeling and analyses completed to date. Seepage modeling to evaluate seepage impacts on embankment design was performed using SEEP/W by Geo-slope

International. Monitoring results from the Test Cell Program were used to calibrate the hydraulic conductivities in the two-dimensional finite element model. This work is reported in the Reservoir Seepage Analysis Technical Memorandum (Appendix 8-10). Given the wide variation in the geology, these conductivities were deliberately derived to represent the broad characteristics of the strata and averaged over depth.

The results of SEEP/W were further refined using MODFLOW, a three-dimensional finite difference model. The parameters shown in Table 8.7-1 have been adopted at this stage but are subject to review as results of the soil investigation and testing are received.

Although the model results fitted the Test Cell data, the solution under these parameters was very sensitive to the conductivities in the caprock, Fort Thompson Formation, and the Caloosahatchee Formation.

Table 8.7-1 Test Cell Hydraulic Conductivities

Stratum	k_h (feet/day)	k_v (feet/day)
Muck (not modeled)		
Caprock	100	1
Fort Thompson	500	10
Caloosahatchee	400	8
Tamiami	36	18
k_h = horizontal hydrostatic conductivity k_v = vertical hydrostatic conductivity		

8.7.4 Recommended Foundation Seepage Control

In view of the potential for piping, an embankment foundation cutoff wall to a minimum depth of 26 feet or the 34 foot base of the Fort Thompson Formation is recommended for seepage control. This is a minimum protection to ensure stability for the general embankment profile. Issues related to EAA Reservoir A-1 water loss are described in Section 9.

The embankment design should make allowance for possible settlement of the material in the cutoff trench. It is recommended that further evaluation of the relative modulus between a cutoff wall and RCC dam will be required if the RCC concept design is advanced to the next stage of design.

A shallower key trench type cutoff is suitable for the embankment section adjacent to STA-3/4 and the east side of the Holey Land Tract which abuts the EAA Reservoir A-1 due to the downstream hydraulic conditions. The embankment cross section proposed in this area makes use of the existing STA-3/4 feeder canal embankment and seepage collection canal to extend the length of the seepage path exit and to provide a balancing head against exit pressure.

The advantages of a shallow pressure relief cutoff trench at the downstream toe of slope should be investigated as the design is refined.

8.7.5 Perimeter Seepage Collection

The perimeter seepage collection canal should have stable side slopes and the depth should be compatible with seepage collection operation and maintenance requirements. The canal should have the minimum depth and width possible to limit excavation quantities of the sandy silt/silty sand materials present in the Fort Thompson Formation.

8.8 FOUNDATIONS

A gravity RCC dam section with limited foot print would apply higher loads to the foundation. A gravity dam should be sized to limit applied pressures to acceptable limits with an appropriate factor of safety. Special measures are anticipated to be required at this site to strengthen some foundation areas. Predicted strengthening may include excavating thin caprock and replacing it with competent concrete or constructing a widened base of concrete in areas where caprock does not exist. When the embankment crosses local features such as the existing canals special cleaning and backfill will be required to avoid differential movement. Foundation bearing capacity is not a significant consideration for an embankment cross section at this site.

8.9 SETTLEMENT

8.9.1 Foundation Settlement

The most compressible material in the existing ground is the organic peat surface layer. This layer will be removed from the foundation prior to embankment construction. Materials beneath the peat are expected to deform elastically with minimal long-term residual movement under the stress of an embankment. It is not considered necessary to make allowance in the embankment height for settlement of the foundation.

8.9.2 Embankment Dam

The materials comprising the maximum height section of the embankment consist of random excavation and “raked” random fill materials from the Fort Thompson Formation. These materials consist of rock pieces (up to 15 percent) and gravel and shells mixed with predominantly sandy silts and silty sands. At the moisture contents and densities anticipated after construction, it is not considered necessary to make significant allowance for settlement of embankment materials.

8.9.3 Gravity Dam - RCC

The gravity dam will be stiff relative to the foundation and will apply higher contact pressures to the foundation than an embankment section. Variation in foundation stiffness could result in differential settlement. Significant movement could open a preferential seepage path at the contact between the dam and its foundation.

8.10 BORROW

8.10.1 General

Material resources to support construction of either an earth filled embankment or RCC dam (excluding cement and additives) are available on site. The aggregate volumes and processing requirements will depend on the type of embankment chosen.

8.10.2 Gravity Dam

8.10.2.1 Concrete Aggregate

For the gravity dam, the caprock/upper limestone is suitable as the source of aggregate for concrete. Blasting is required to break up the caprock/upper limestone section to suitable size for crushing. Washing will be required to produce materials of the required gradations and quality. It is anticipated that the primary borrow areas for aggregate production will be located within the

EAA Reservoir A-1 where caprock is thickest. The contractor will optimize his blasting pattern to facilitate his crushing operation.

8.10.2.2 Cement

Cement for RCC must be imported to the site and properly stored until use. For the RCC alternative, the amount of cement for the project is anticipated to exceed 240,000 tons.

8.10.3 Embankment

8.10.3.1 Rockfill

Material for the rockfill can be obtained from the layer of caprock/upper limestone existing immediately below the surface soils. This layer would be excavated from the seepage canal and is available in borrow area locations throughout the EAA Reservoir A-1 area as needed. Blasting is required to adequately break up this layer for fill material use. The blasting pattern should be selected such that rockfill is produced at the optimum gradation for direct use without processing.

It is currently planned that the blasted material will be hauled to the embankment location and stockpiled either on the interior bench between the embankment and the internal borrow area, or in the location of its final placement in the embankment.

8.10.3.2 Random Fill

Material excavated from the Fort Thompson Formation immediately below the caprock/upper limestone will serve as the source for random fill. In the central zone of the embankment, rock fragments larger than six inches will be removed to develop the water detention zone (water barrier) of the embankment. This sorting will occur on the embankment after initial spreading and before compaction using a “rock rake”. This material is readily available beneath the caprock/upper limestone in all site excavations.

Of importance to the cost of this material, is the presence of two layers of limestone within the upper 15 feet of the Fort Thompson Formation. These limestone layers were noted to be of low strength and could be removed with an excavator. Additional handling or raking will be required to remove the larger limestone pieces from the central random fill material zone of the embankment.

8.10.3.3 Drainage Materials

Drainage materials will be obtained by crushing, screening, and washing the excavated caprock/upper limestone to the specified gradation. Since the preparation of the filter and drain materials require the use of a crusher, the source of materials is expected to be the interior borrow areas.

8.10.3.4 Roller Compacted Concrete (RCC)

RCC will be obtained from a central batching plant as described for the concrete gravity dam above. Aggregates can be obtained by processing on-site rock materials.

8.10.3.5 Topsoil

In accordance with SFWMD Design Standards, a layer of topsoil is to be added to the exterior face of an embankment prior to seeding. Area practice is that this topsoil material is obtained from the local peat, and is available from the strip and material removed from the embankment construction area. The peat can be stockpiled adjacent to the location of the exterior toe of embankment to reduce handling and cost.

8.11 EMBANKMENT SECTION ALTERNATIVE EVALUATIONS

The evaluation of embankment sections included the conceptual development of embankment and RCC dam sections based on information obtained from the Test Cell Program and discussions with the USACE staff, Jacksonville District. The initial evaluation began with the development of the two embankment sections constructed for the Test Cell Program. From the lessons learned during the Test Cell Program construction and monitoring, six alternative embankment sections were developed to evaluate a different combination of material zoning and seepage control concepts. Three alternative RCC dam sections were also evaluated at that time through communications with the USACE staff. Ongoing evaluations, analyses, and input from the attendees of the Criteria Committee Meeting held in June, 2005 resulted in adding an additional embankment concept and the evaluation of the impacts of a wave break bench on the interior slope (or face) of the perimeter embankment.

The crest height for each section has been referenced to as the height above OG. This reference was used for the evaluations but a crest elevation will be designated for the embankment design following selection of the final embankment cross-section.

The results of the evaluations completed to date are summarized in the following section of the BODR. A detailed presentation of the evaluations and analyses are provided in Appendix 8-1.

8.11.1 Alternative Embankment Sections

8.11.1.1 Embankment Alternative with Upstream Rockfill Section

An Embankment Alternative with Upstream Rockfill Section is presented on Figure 8.11-1. This alternative is considered the baseline embankment section for the comparative evaluations. This embankment alternative was developed to utilize materials from the required seepage collection canal excavation and borrow excavations with minimum material sorting and processing. The upstream rockfill section will be produced from the caprock/upper limestone. The random fill consists of unsorted rock pieces (less than 18 inches maximum size) and silty sand placed without sorting or processing. A transition zone is included between the rockfill and random fill zones to prevent migration of fines from the random fill into the rockfill due to gravity action or water movement during EAA Reservoir A-1 drawdown or changes in EAA Reservoir A-1 water level. The processed random fill zone (watertight barrier) between the transition zone and vertical chimney is to be processed on the fill by raking to eliminate all rock pieces larger than six inches prior to compaction. The top of this watertight zone extends up to the NWL plus the PMP. The vertical chimney is provided for internal drainage, to protect against internal erosion of fines within the random fill, and to control the phreatic line in the downstream random fill zone.

A horizontal blanket drain extends over the caprock to relieve seepage pressures and control piping of fines from the foundation. The horizontal drain discharges into a granular toe drain at the downstream toe of the embankment. Discharge from the toe drain will be concentrated in low areas of the caprock surface. Top soil (using muck or peat stripped from the embankment foundation) and seeding is provided on the downstream slope. Upstream slope protection is provided by RCC using flat plate construction on the 3H:1V slope extending to the level of the

NWL plus the PMP. RCC stepped construction is used above this water level to the crest to provide added wave breaking protection.

Foundation preparation for this alternative includes blading the caprock surface to remove muck and clay remaining after stripping, and brushing the caprock surface using a power broom. The soil-bentonite cutoff will be located generally beneath the center of the embankment section and extended a minimum of three feet above the caprock surface into the watertight zone between the rockfill and chimney drain. The cutoff trench will be widened through the caprock to allow placement of a lean concrete seal on each side of the cutoff.

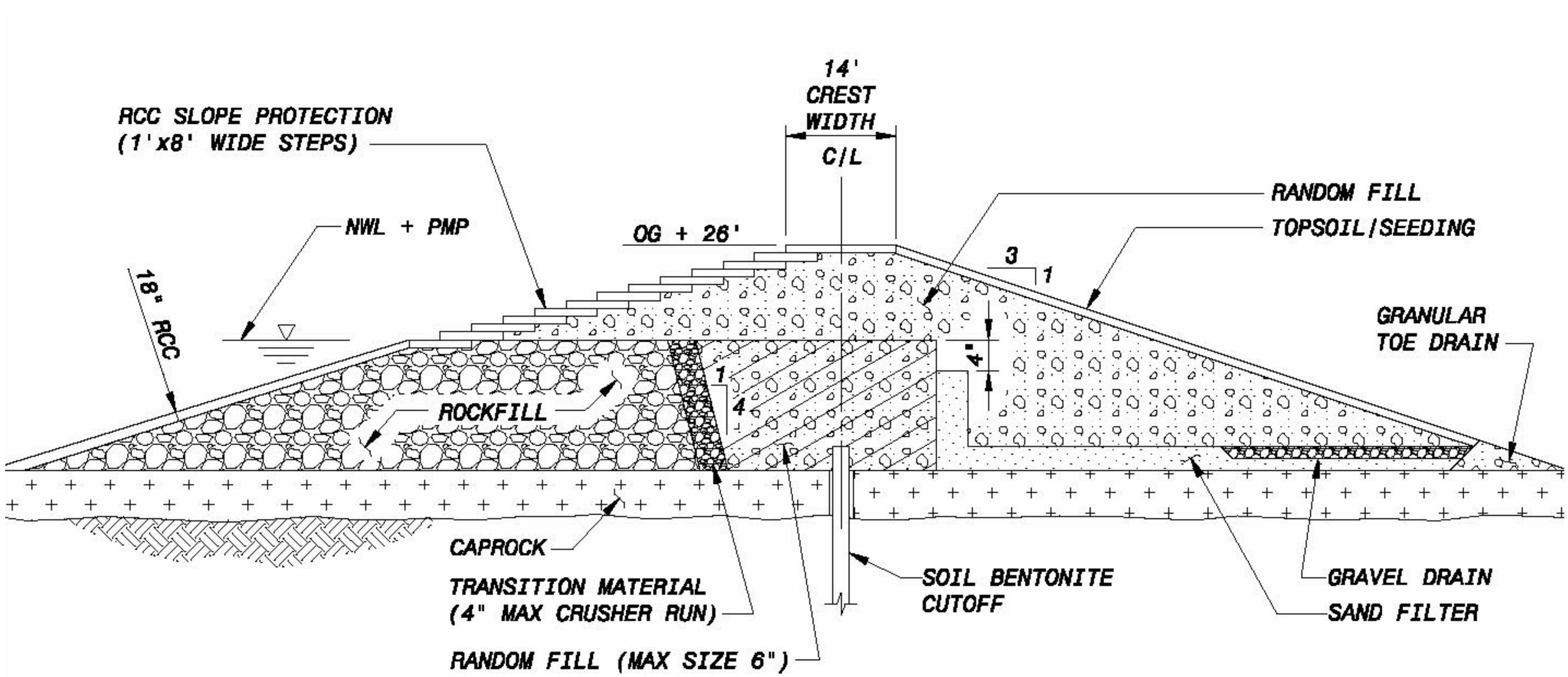
8.11.1.2 Embankment with Geomembrane

This embankment alternative is presented in Figure 8.11-2. The embankment contains a sloping geomembrane to provide a watertight barrier in the upstream portion of the random fill embankment. Select fill obtained by selective excavation of silty sand is shown to encompass and protect the geomembrane. The geomembrane is extended over the top of the soil-bentonite cutoff to provide continuity of the seepage barrier. Transition material zones are provided to separate the select fill and random fill zone. Blanket filter and drain sections are placed over the caprock foundation for seepage and piping control. This alternative includes a finger drain arrangement to direct the seepage to the downstream toe drain. Upstream and downstream slope protection is similar to Alternative 1. Foundation preparation includes cleaning of the caprock surface with a power broom.

8.11.1.3 Embankment with Central Core and Shallow Core Trench

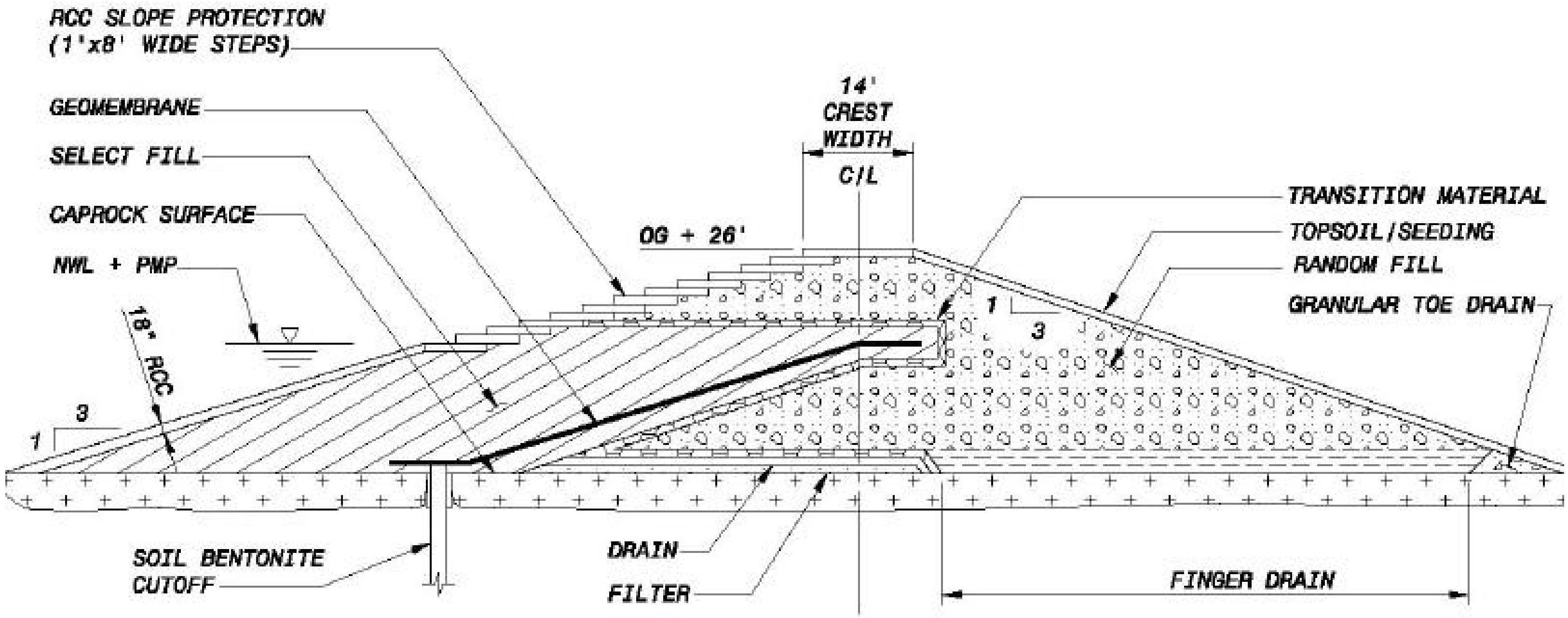
This alternative includes a central select fill zone as the watertight barrier in the embankment with a shallow extension through the caprock and into the underlying silty sand. The alternative is shown in Figure 8.11-3. The select fill consists of silty sand material with a maximum particle size of four inches from the required excavations. The bottom width of the select fill zone is shown at about 1.5 times the water head defined by NWL + PMP. A shotcrete seal through the caprock is provided to fill voids and to mitigate piping. Random fill zones provide additional stability upstream and downstream of the select fill zone. The select fill zone is extended to the level of NWL + PMP. Transition material consisting of four inches minus crusher run caprock/upper limestone is used to cap the random fill and select fill zones, and to provide a base for the RCC slope protection. The chimney, horizontal filter and drain, and toe drain are similar to the previous sections. Foundation preparation for this alternative includes power brooming of the stripped caprock surface.

Figure 8.11-1 Embankment Alternative with Upstream Rockfill Shoulder



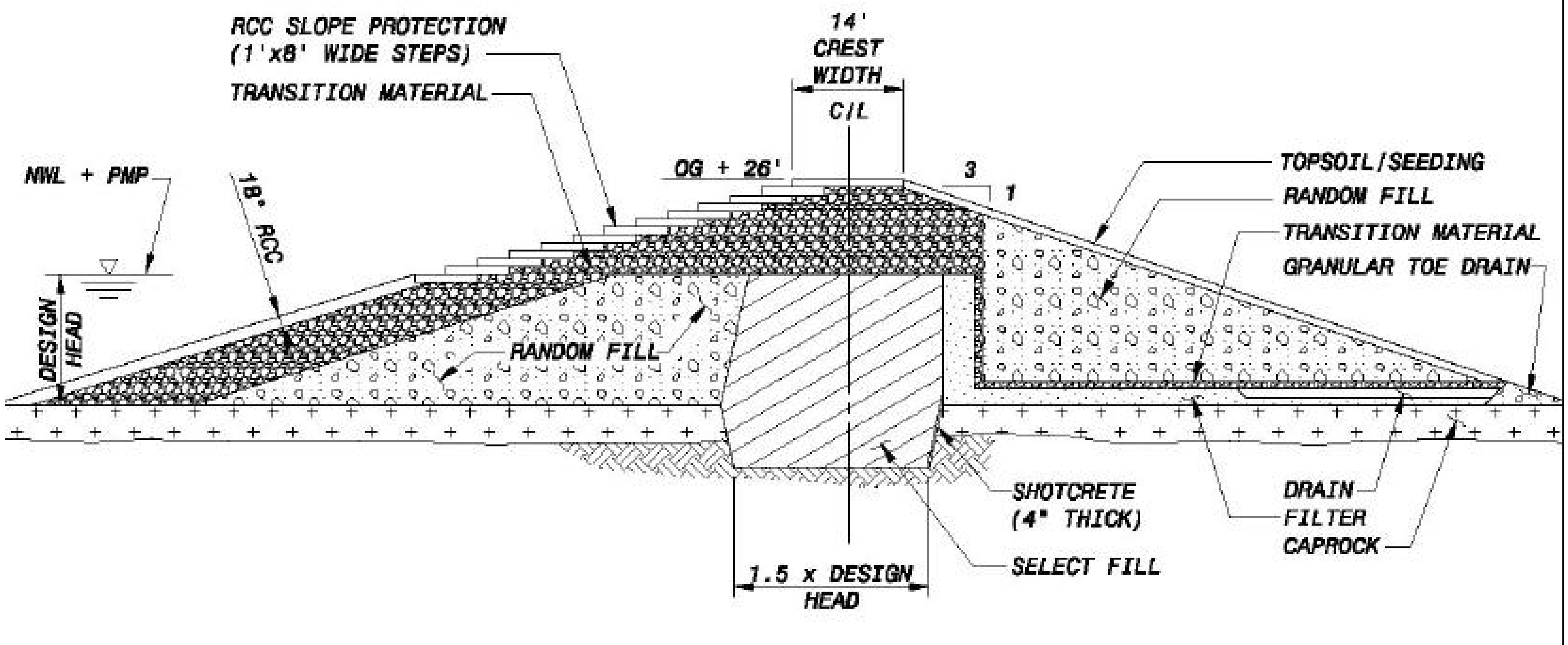
No scale

Figure 8.11-2 Alternative 2 Embankment with Geomembrane



No scale

Figure 8.11-3 Alternative 3 Embankment with Central Core and Shallow Core Trench



No scale

8.11.1.4 Roller Compacted Concrete (RCC) Gravity Dam

An RCC dam section proposed by the USACE is presented in Figure 8.11-4. As designed by USACE, the section is 27 feet high above foundation support level (current wave modeling suggests that a 28-foot height would be required), vertical upstream face with the downstream face stepped resulting in three, nine foot vertical faced steps. The base width of the section is 17 feet and the crest width is 12 feet.

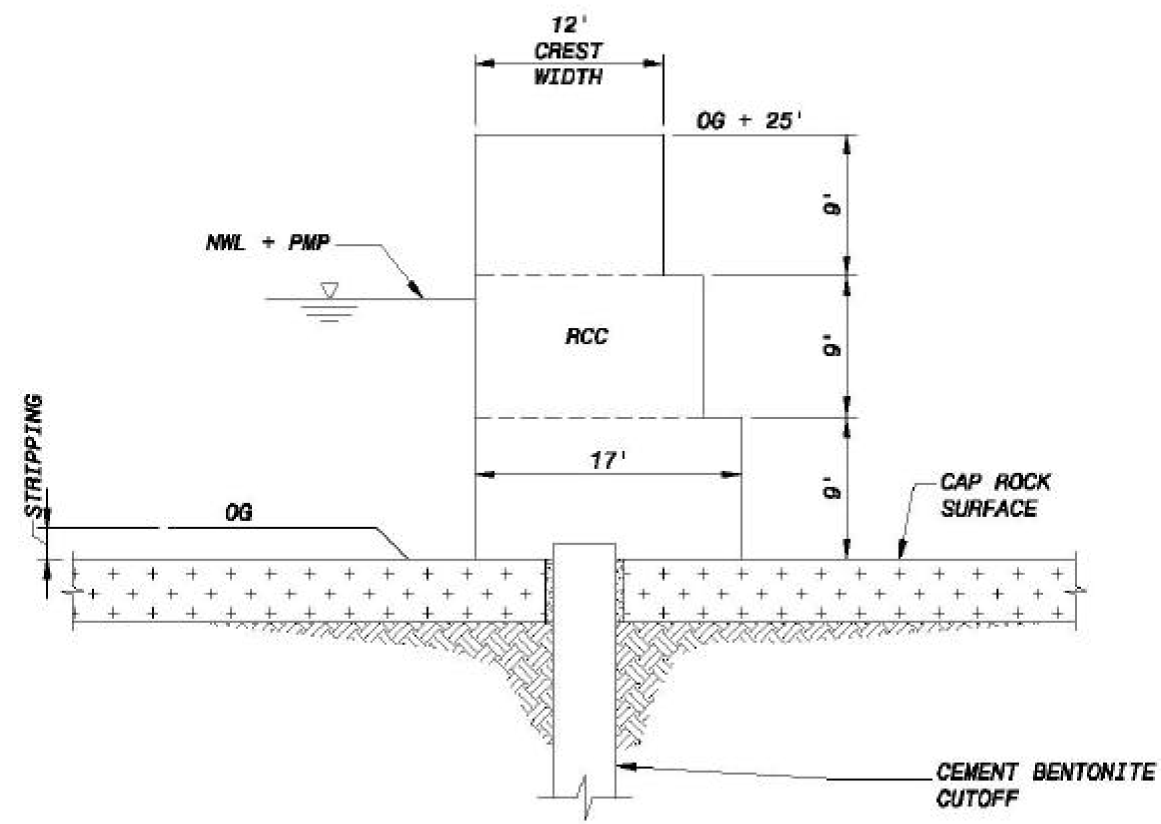
Based on observations from the Test Cell Program, foundation preparation for an RCC dam at this site would, as a minimum, consist of power cleaning with high pressure air, water, or an air/water combination. Cleaning solution holes in the caprock to a minimum depth of four inches would be required to prepare the surface for treatment. The depressions should be filled with a sand/cement grout or fine-aggregate concrete to adequately fill and seal the depressions prior to RCC placement.

Both the upstream and downstream faces will be formed to confine the RCC during placement and compaction. Forming may be externally braced or restrained using embedded ties into the RCC placement. The requirement for a conventional concrete or grout enriched facing zone to provide a watertight barrier should be evaluated if the design concept is advanced in the next phase of the design process.

Based on foundation conditions observed at the Test Cell sites and results of recent boring data across the site area, special foundation treatment beneath the footprint of the RCC section should be expected due to the variability of foundation conditions anticipated. An advanced definition of the magnitude and scope of appropriate foundation treatment is difficult to estimate due to the lack of continuous subsurface data along the projected embankment alignment. Based on present observations and assessments, it is anticipated that foundation treatment may include removal of thin (less than three feet) caprock segments and replacement of the excavated materials with conventional lean concrete (2,000 psi design strength). The magnitude of foundation treatment and probable cost is not included in this BODR. Additional data from the current investigations will be reviewed for that assessment at a later date.

The cutoff beneath the RCC dam may require a “plastic concrete” (See Figure 8.11-4) design to prevent wall backfill separation from the rigid RCC structure. The construction techniques for installation of a plastic concrete cutoff are significantly more costly than for a soil-bentonite cutoff.

Figure 8.11-4 Alternative No. 4 Roller Compacted Concrete (RCC) Dam – (USACE)



No scale

8.11.1.5 Concrete Faced Rockfill Embankment

This embankment configuration is presented in Figure 8.11-5 The embankment consists of an upstream, reinforced concrete face supported on compacted rockfill. The upstream concrete face provides the required watertight barrier. The upstream slope may be as steep as 1.6H:1V to minimize the facing volume. Special forming and screed equipment is required for concrete placement on the steep slope. The concrete face is cast on a fine processed stone leveling layer to prevent loss of concrete paste into the rockfill and to act as a filter. The face slab is connected to the foundation and supported at the toe by a reinforced concrete cap or plinth. The cutoff would be placed beneath the concrete cap to provide a watertight seal. The cutoff should be constructed of “plastic concrete” to limit the settlement of the cutoff fill and the development of a gap at the top of the cutoff. Provisions for backup grout sealing to fill any gap that may develop after settlement of the “plastic concrete” may be required. A reinforced concrete parapet wall is provided at the top of the upstream slope to mitigate wave overtopping, thereby reducing the structural height of the embankment. Conceptual details of the concrete toe cap, grout seal at the top of the cutoff, and parapet wall are shown on Figure 8.11-6. The downstream slope is 3H:1V to accommodate mowing and maintenance equipment. A downstream zone of random fill has been included in the embankment section to provide for use of random fill encountered in the seepage collection canal and borrow area excavations.

A downstream horizontal drain blanket of sand and gravel is provided to safeguard against piping through the foundation caprock. Foundation preparation for the CFRD would be limited to stripping and power broom cleaning, except for power cleaning and grout treatment beneath the concrete toe cap.

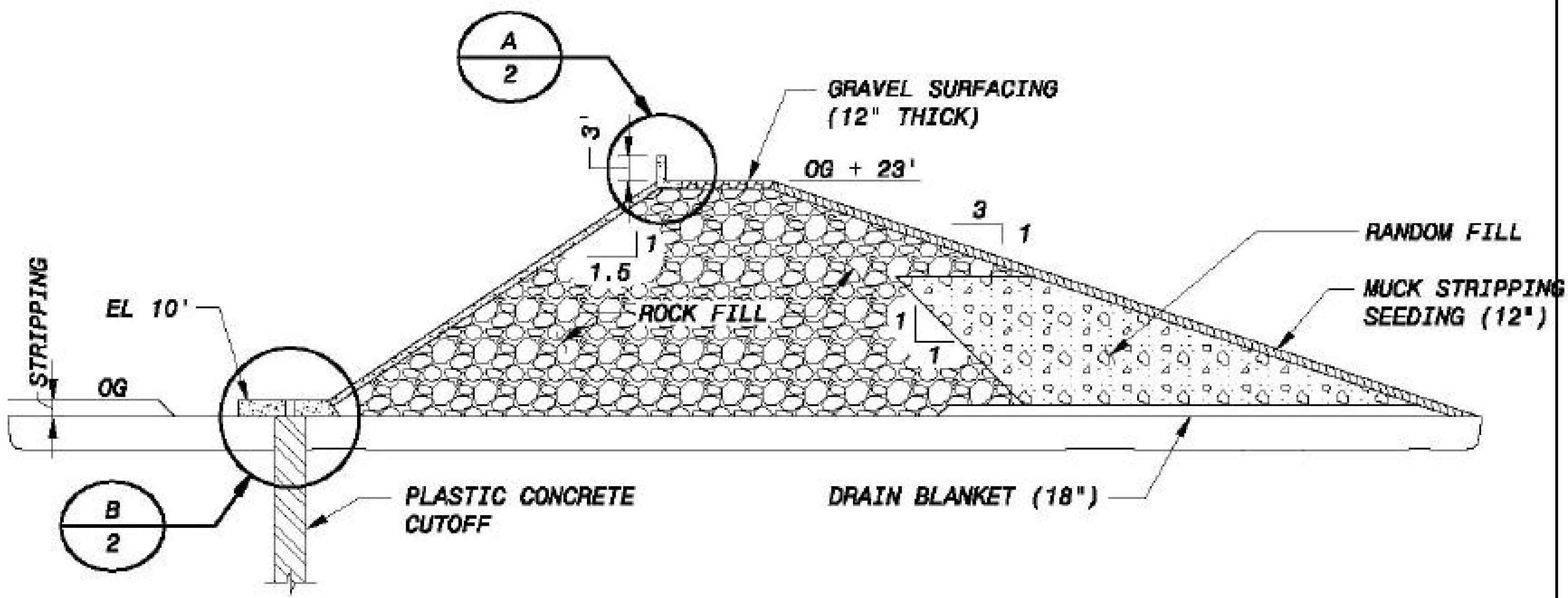
8.11.1.6 Embankment Section Adjacent to the STA-3/4 Supply Canal Levee

An alternate section for the levee along the STA-3/4 Supply Canal is presented in Figure 8.11-7. This alternative includes incorporating the existing levee into the downstream toe of the EAA Reservoir A-1 embankment thereby reducing the required embankment volume.

The requirement for an internal drainage system to control piping has been eliminated due to extended seepage path and “back pressure” provided by the water stage in the Supply Canal. The existing seepage canal paralleling the existing levee on the interior, or EAA Reservoir A-1 side, can be filled with compacted silty sand to cutoff the horizontal seepage path through the caprock and the shallow caprock/silty sand foundation material interface. If necessary, downstream pore water pressures and seepage quantity into STA-3/4 could be reduced further by constructing a cutoff wall from the bottom of the Supply Canal's seepage canal.

This reduced downstream embankment section could be adapted to any of the alternative embankment sections presented.

Figure 8.11-5 Concrete Faced Rockfill Embankment



No scale

Figure 8.11-6 Concrete Faced Rockfill Embankment Toe and Parapet Details

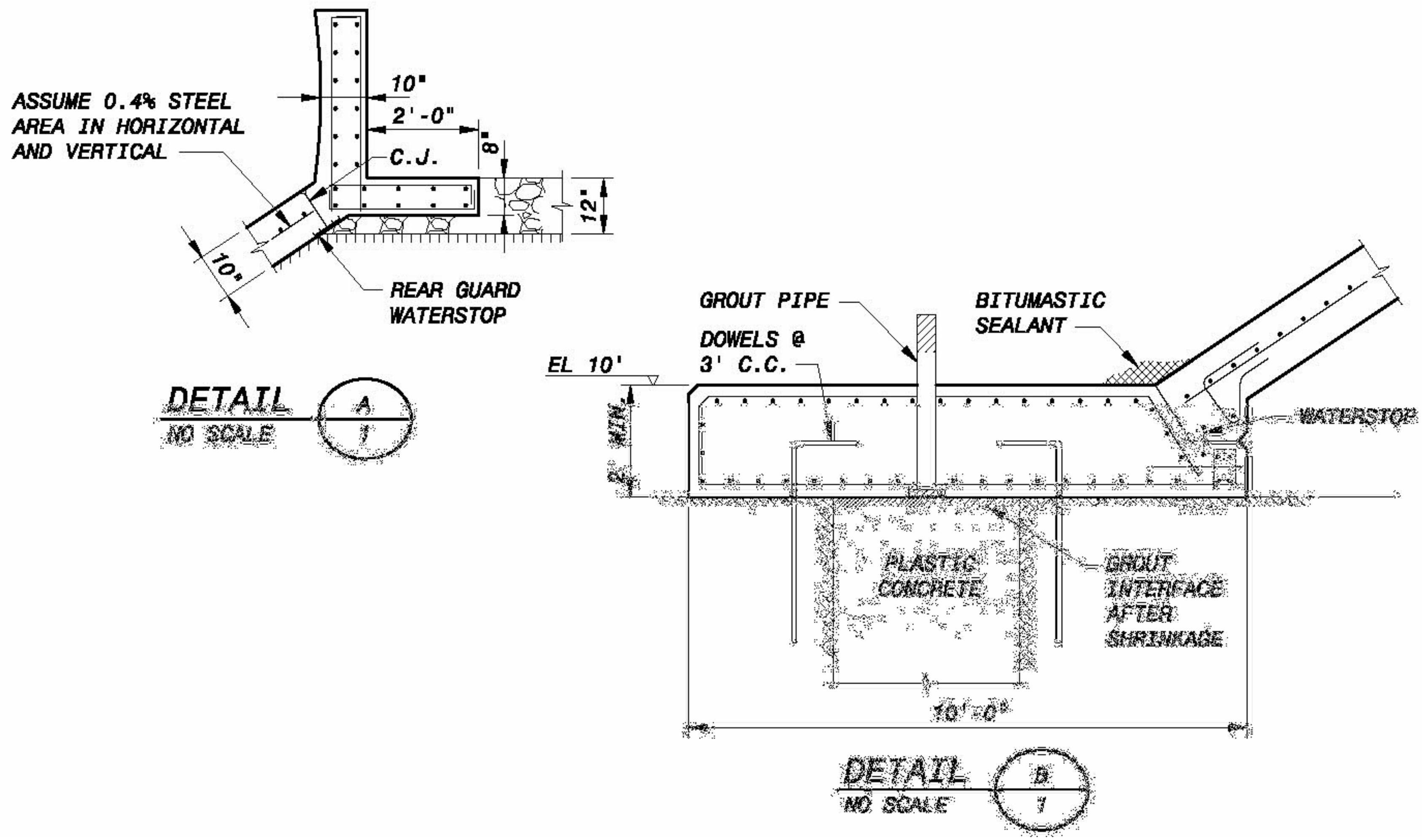


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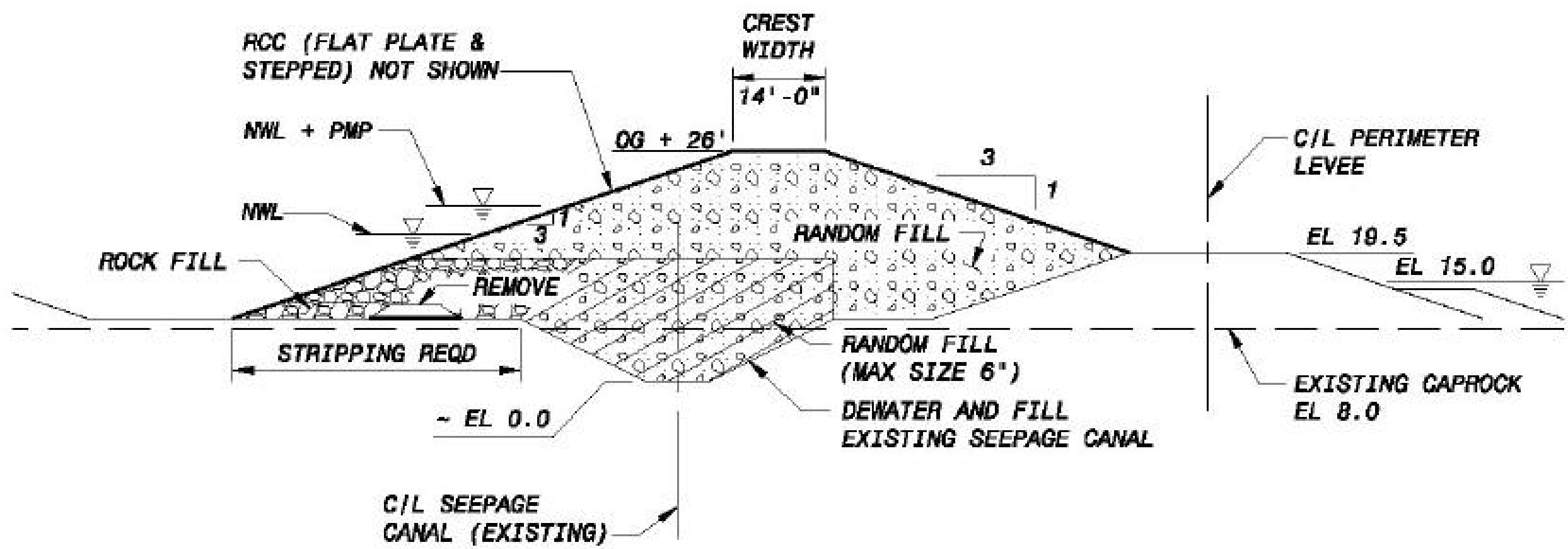
Figure 8.11-7 Embankment Section Adjacent to STA-3/4 Supply Canal Levee



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8.11.2 Screening of Embankment Section Alternatives

The embankment alternatives described previously were screened considering the design standards and probable costs. These sections are:

- Embankment with rockfill upstream section (crest at OG+26 feet)
- Embankment with geomembrane
- Embankment with central core
- RCC gravity dam
- Concrete-faced rockfill embankment

The embankment with an upstream rockfill section was selected as a base option against which to compare the other sections. The advantages and disadvantages of the options relative to the base are presented in Table 8.11-1.

The advantages of the base (embankment with upstream rockfill section) design include:

- All materials from the required excavation of seepage collection canal can be used in the embankment
- The material suitability of available on-site materials and construction process has been confirmed in the Test Cell Program
- Stripping to the caprock surface and cleaning using a rotating power broom is adequate foundation preparation beneath the embankment except in local caprock areas
- Apart from the cement for the slope protection and bentonite for the cutoff wall, all materials for embankment construction can be produced on site
- Processing of excavated material for embankment construction can be limited to crushing, screening, washing, and blending for drain materials, riprap bedding and aggregates for RCC slope protection
- The embankment has the flexibility to accommodate settlement in the foundation
- Earthfill embankments are very resilient to earthquake loading
- The embankment deformation and slurry cutoff wall settlement are compatible thereby maintaining the interface connection
- SFWMD is familiar with the maintenance requirements for embankments
- The seeded downstream slope blends in with the general surroundings and adds aesthetic value
- Least cost

Table 8.11-1 Evaluation and Opinion of Probable Cost Comparison

Embankment Section	Embankment	Embankment with Geomembrane	Embankment with Central Core and Shallow Cutoff	RCC Dam	Concrete Faced Rockfill Embankment
Location of Drawing	Figure 8.11-1	Figure 8.11-2	Figure 8.11-3	Figure 8.11-4	Figures 8.11-5 and 8.11-6
Relative Advantages	Base design.	No chimney drain required		Occupies less area than an embankment, so EAA Reservoir A-1 storage can be maximized. Minor overtopping may be acceptable without damage. Downstream slope maintenance generally not required if defect free. Risk of piping in dam eliminated. Avoids the need to use moisture sensitive Fort Thompson Formation material.	Risk of piping in dam reduced. Potential to employ locally more crafts.
Relative Disadvantages	Base design.	Geomembrane adds cost but adds little decrease in seepage rate. Increased requirement for imported materials. Embankment supporting membrane must not be vulnerable for differential movement. Joints in membrane can be vulnerable. Connection to cutoff wall requires special detail. Working in high winds or bad weather is difficult. Increase in quality control requirements.	Wide cutoff trench excavation requires dewatering. Production of the separate Select Fill zone requires additional stockpiling and material drying. Top of exterior random fill zone is narrow. Large volume of processed random fill material is required. Omission of a deeper cutoff increases seepage rates and risk of foundation stability problems.	Special foundation treatment required where caprock does not exist or where the caprock is determined to be thin. Predicting the magnitude of cracking and control of crack location with certainty is difficult due to the variable foundation conditions. Seepage is expected to occur through cracks. Foundation preparation by air/water blasting will be required. Settlement of the cutoff must be compatible with the rigidity of the RCC structure. Supply risk because a large volume of cement must be imported. Requires larger volume of processed aggregates than an embankment. Temperature control of the RCC components required in hot temperatures. Moisture control in stockpiled aggregates difficult during wet season. Utilizes less volume of the required seepage collection canal excavation, therefore spoil volume is increased. An RCC structure does not fit aesthetically with the surroundings. EAA Reservoir A-1 is less attractive for recreational use. Vertical upstream face creates a safety hazard; means for escape required.	Upstream plinth is considered critical for performance. Treatment required where caprock does not exist or where the caprock is determined to be thin. Foundation preparation by air/water blasting will be required. Settlement of the cutoff must be compatible with the rigidity of the concrete plinth structure. Steeply inclined upstream face creates a safety hazard; means for escape required. Upstream wave wall restricts width of crest for maintenance operations. Vulnerable to poor execution of intricate waterproofing details. Requires steel reinforcement to be imported to site. Risk of supply problems increased. Increased safety risk due to working at height on reinforced concrete face.
Opinion of Probable Total Cost					
Construction Cost(3)	$ 403.5M(1)	$442.0M(1)	$593.3M	$727.2M(2)	$584.0M
Differential Probable Total Cost (increase over base cost)	Base	$38.5M	$189.8M	$323.7M	$180.5M
Reference	Table 8.13-1	Table 8.13-2	Table 8.13-3	Table 8.13-4	Table 8.13-5

Notes: (1) Soil-bentonite cutoff (2) Plastic concrete cutoff (3) All Probable Construction Cost includes a contingency of 30 percent per requirement of BODR phase estimates.

The disadvantages of this design include:

- The silty sand material from the Fort Thompson Formation, which will be used as random fill, is very wet and difficult to work when taken directly from the excavation. However, experience from the Test Cell Program and SFWMD's levee construction experience has demonstrated that the materials can be used successfully when allowed to dry or drain
- Armored slope protection, such as RCC, will be required for wave protection unless a wave break is provided
- Requires more frequent surveillance of upstream slope protection
- Requires more instrumentation for surveillance/monitoring than a RCC gravity dam
- The potential for internal erosion and piping is higher than for a RCC gravity dam
- Overtopping is not allowed

8.12 EMBANKMENT SECTION MODIFICATIONS BASED ON WAVE ANALYSES

Additional wave height analyses have been completed following the draft preparation of the Embankment Technical Memorandum, and the Criteria Committee Meeting held in June, 2005. These additional analyses were introduced by the USACE in their evaluation for preparation of the PIR for the EAA Reservoir A-1 Project. The additional wave analyses included assessment of the impact of an interior wave break bench on the height of the embankment and the impact of the wave configuration (monochromatic or regular shape, and irregular wave shape) on the freeboard or superiority requirements for full containment. The analyses of the impact of waves and storm surge on the embankment are described in Section 5 and Appendix 5-14.

A discussion of the impacts of the wave analyses on the embankment and RCC (gravity) dam heights are presented in the following subsections. The embankment heights presented have been developed to provide full containment of the generated wave including wave run-up and wind surge as defined by DCM-2. An evaluation and cost summary are presented in Table 8.12-1.

8.12.1 Embankment

At the time of analysis the governing design case was a wind speed of 158 mph considering the regular wave form and an EAA Reservoir A-1 pool level at NWL (OG+12). An embankment constructed to a height of 25.7 feet above original ground (OG+25.7) and containing a 3H:1V upstream slope of stepped RCC was required (Figure 8.11-1).

The DCM governing this aspect of the design has been changed since the analyses were originally carried out and is still the subject of on-going discussion within the USACE. The results of the analyses remain despite changes to the ruling criteria. The current revision of the DCM eliminates the 158 mph wind as a design case and defines it as a sensitivity case. Under the current criteria a 103 mph wind combined with the PMP is to be considered the design case. Under an irregular wave form not more than 0.1 cfs should pass the top of slope as described in Section 5. An embankment height of 25.5 feet above ground (OG+25.5) would be required to meet this criterion. Within the accuracy of the model, this is essentially the same result as the 158 mph case.

Table 8.12-1 Comparison of Wave Protection Options and Probable Cost

Type	Wave Form	Wind Speed (mph)	Reservoir Level	Required Crest Height (ft)	Cost Comparison [3]	Comment
Embankment [1]	Regular	158	NWL	OG+25.7	$400M	
		103	NWL+PMP	OG+24.6		
	Irregular (0.1 cfs overwash)	158	NWL	OG+27.5		
		103	NWL+PMP	OG+25.5		Design case
Embankment with wave break bench at OG+16 feet [1]	Regular	158	NWL	OG+22.4	$410M	
		103	NWL+PMP	OG+21.9		
	Irregular (0.1 cfs overwash)	158	NWL	OG+22.0		
		103	NWL+PMP	OG+22.0		Design case
RCC Dam [2]	Regular	158	NWL	OG+27.5	$730M	
		103	NWL+PMP	OG+26.5		
	Irregular (0.1 cfs overwash)	158	NWL	OG+28.0		
		103	NWL+PMP	OG+27.5		Design case
RCC with wave break bench at OG+16 feet [1]	Regular	158	NWL	OG+22.4	$590M	
		103	NWL+PMP	OG+22.0		
	Irregular (0.1 cfs overwash)	158	NWL	OG+22.0		
		103	NWL+PMP	OG+22.0		Design case

Notes: [1] Soil-bentonite cutoff
[2] Plastic concrete cutoff
[3] Opinion of Probable Construction Cost includes a contingency of 30 percent per requirement of BODR phase estimates. Costs in Table 8.12-1 were interpolated or extrapolated from data shown in Tables 8.13-1 to 8.13-9

The construction of a wave break bench at OG+16 on the upstream slope (Figure 8.13-1) would result in a lower required embankment height as described in Section 5. The results of the wave analyses under these conditions are summarized, with costs, in Table 8.12-1. On balance the wave break bench is not cost effective.

The embankment with the upstream rockfill section and a crest height of OG+26 without a wave break bench is evaluated to be the most cost effective alternative for the embankment section.

8.12.2 RCC Dam

An RCC dam constructed to a height of 25 feet above original ground with a 2.5 foot high parapet wall, and containing a vertical upstream face will provide full containment for a wind speed of 158 mph considering the regular wave form and an EAA Reservoir A-1 level at NWL.

A three foot parapet wall above the crest of the RCC dam at OG+25 would be required to provide full containment considering the irregular wave form and the 158 mph wind speed. This represents an additional height increase of 0.5 feet to provide containment prescribed in DCM-2.

Constructing a 15-foot wide wave break bench at OG+16 against the upstream vertical face (Figure 8.13-2) would allow the crest level of the RCC dam to be lowered as shown in **Error! Reference source not found.**. Adding the bench would allow the cutoff to be relocated from beneath the RCC section to beneath the bench fill. This relocation would allow the cutoff to be constructed as a soil-bentonite wall instead of the more rigid plastic concrete wall.

The RCC Dam options are more expensive than the zoned embankment options.

8.13 SELECTION OF EMBANKMENT ALTERNATIVE AND PROBABLE CONSTRUCTION COSTS

Based on the comparison of probable construction costs, the embankment with the upstream internal rockfill section represents the least cost alternative (Figure 8.11-1). The opinion of probable cost for this alternative includes a 30-foot cutoff for a distance of 12.95 miles of the perimeter. The embankment section parallels the existing Supply Canal levee for the remaining 8.75 miles of the perimeter. The alternative section shown in Figure 8.11-7 would be constructed adjacent to the Supply Canal.

This alternative provides effective wave protection using RCC flat plate and stepped construction. The upstream rockfill section provides back-up protection should cracking occur. The rockfill section also provides additional protection against a catastrophic breach should damage occur to the slope protection.

Based on the results of the technical and cost evaluation, the embankment alternative shown in Figure 8.11-1 is the preferred alternative to be advanced to 30 percent design. The opinion of probable construction cost for this alternative is presented in Table 8.13-1. Tables 8.13-2 through 8.13-9 are the opinions of probable construction cost for the remaining alternatives. A contingency of 30 percent was used for developing costs for the Alternatives in Tables 8.13-1 through 8.13-9. The contingency for developing the project costs for the embankment in Section 23 was 20 percent. (Note: The colors in Tables 8.13-1 through 8.13-9 are intended to make it easier for the reader to view related items and there is no significance to the colors themselves.)

Figure 8.13-1 Embankment with Wave Break Bench for Regular Wave Form

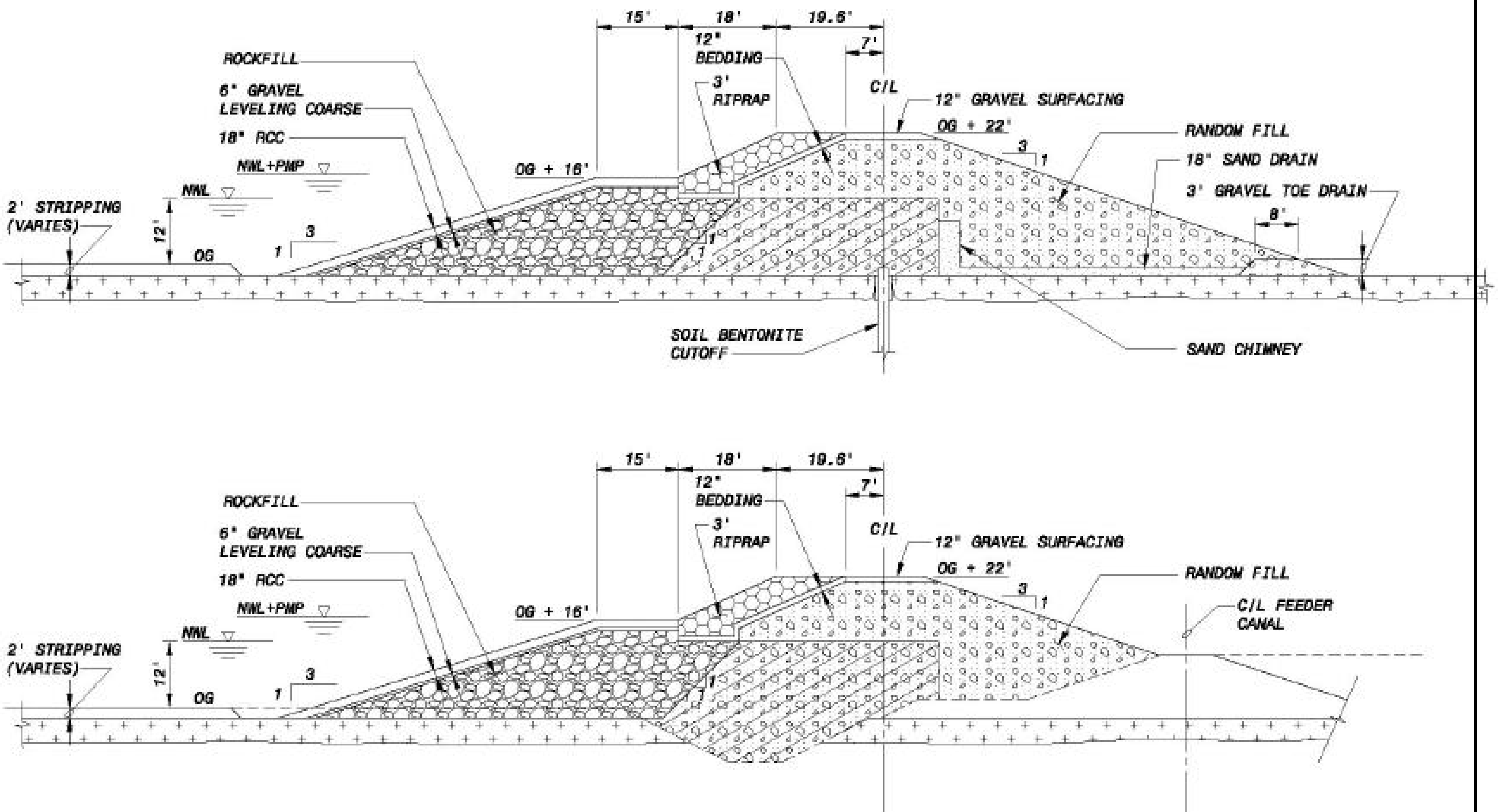

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Figure 8.13-2 RCC Dam with Wave Break Bench for Regular Wave

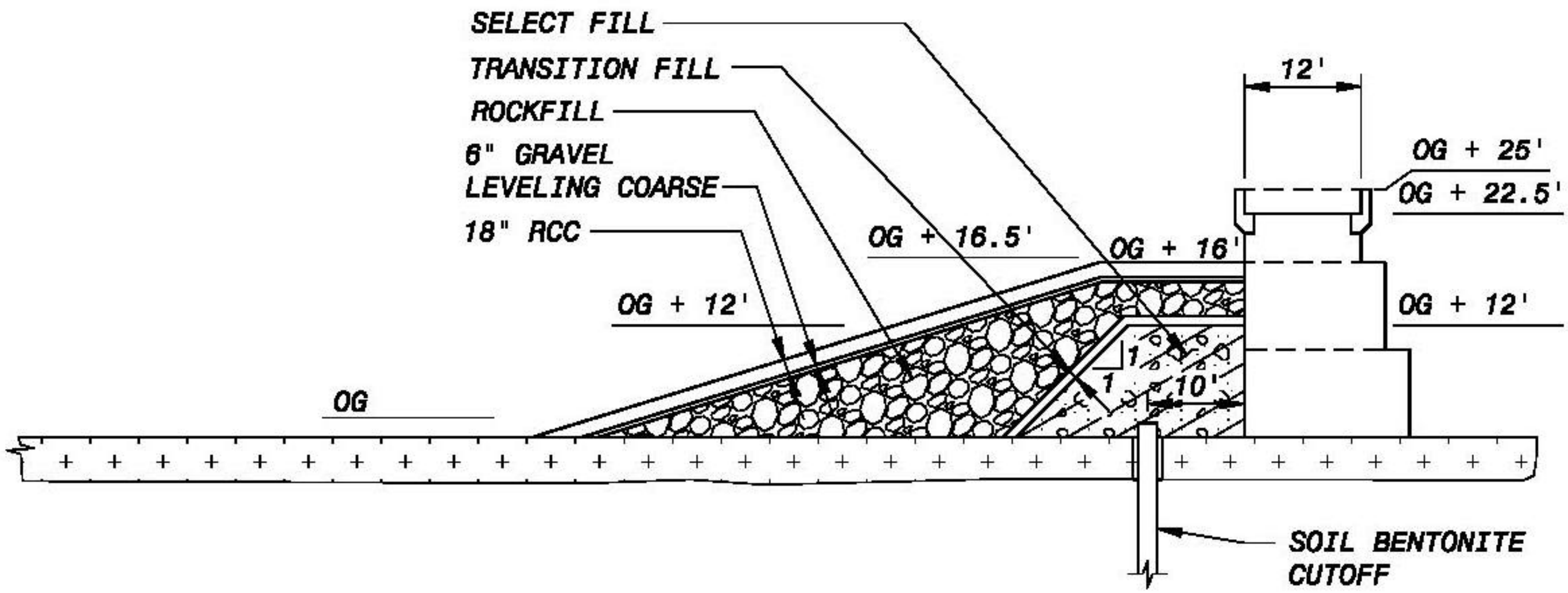

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TABLE 8.13-1

OPINION OF PROBABLE CONSRUCTION COST FOR CASE 1b

Case: 1b

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No.: 141731.0910
Revision No.: 2
Date: 07/26/05

Opinion Of Probable Cost
EAA Reservoir A-1, Up Stream Rockfill Embankment (OG +26), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				55,946	955,274	0	5,976,601	0	0	6,931,875	0.9534	6,609,055	13,540,930
	Subtotal				55,946	$955,274	$0	$5,976,601	$0	$0	$6,931,875		$6,609,055	$13,540,930
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				183,658	3,015,790	2,297,434	12,164,529	0	0	17,477,752	0.9534	16,663,808	34,141,561
	Subtotal				183,658	$3,015,790	$2,297,434	$12,164,529	$0	$0	$17,477,752		$16,663,808	$34,141,561
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.9534	3,440,076	7,048,182
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,440,076	$7,048,182
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				609,596	9,536,983	5,683,347	39,056,936	0	0	54,277,266	0.9534	51,749,558	106,026,825
	Subtotal				609,596	$9,536,983	$5,683,347	$39,056,936	$0	$0	$54,277,266		$51,749,558	$106,026,825
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				103,477	1,707,431	0	1,790,785	0	8,220,329	11,718,545	0.9534	11,172,809	22,891,354
	Subtotal				103,477	$1,707,431	$0	$1,790,785	$0	$8,220,329	$11,718,545		$11,172,809	$22,891,354
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				70,505	1,073,838	5,925,920	1,986,478	5,805,122	0	14,791,358	0.9534	14,102,521	28,893,879
	Subtotal				70,505	$1,073,838	$5,925,920	$1,986,478	$5,805,122	$0	$14,791,358		$14,102,521	$28,893,879
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				40,407	647,012	0	1,375,929	1,316,717	0	3,339,657	0.9534	3,184,128	6,523,785
	Subtotal				40,407	$647,012	$0	$1,375,929	$1,316,717	$0	$3,339,657		$3,184,128	$6,523,785
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				439,113	6,822,119	0	29,421,122	0	927,217	37,170,457	0.9534	35,439,418	72,609,876
	Subtotal				439,113	$6,822,119	$0	$29,421,122	$0	$927,217	$37,170,457		$35,439,418	$72,609,876
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				25,738	392,537	144,172	1,181,367	0	0	1,718,075	0.9534	1,638,064	3,356,140
	Subtotal				25,738	$392,537	$144,172	$1,181,367	$0	$0	$1,718,075		$1,638,064	$3,356,140
11	**Embankment Construction (Cutoff Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				415,861	6,103,947	31,966,704	17,309,216	0	0	55,379,867	0.9534	52,800,811	108,180,678
	Subtotal				415,861	$6,103,947	$31,966,704	$17,309,216	$0	$0	$55,379,867		$52,800,811	$108,180,678
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.9534	159,736	327,273
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$159,736	$327,273
	Total				**1,978,684**	**$30,748,789**	**$46,017,577**	**$113,441,498**	**$7,121,839**	**$9,250,795**	**$206,580,498**		**$196,959,985**	**$403,540,483**

EAA Reservoir A-1 Basis of Design Report

Case: 1b

January, 2006

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
2
07/26/05

Opinion Of Probable Cost

EAA Reservoir A-1, Up Stream Rockfill Embankment (OG +26), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Dee
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	Labor: M-H per Unit	Labor: Man-Hours	Labor: Duration Days	Labor: Average Wage Rate	Labor: Labor Cost	Material: Unit Cost	Material: Material Cost	Equipment: Code	Equipment: No.	Equipment: Avg. Cost ($/hr)	Equipment: Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1		**General Requirements**																			
		Mobilization	1	LS	2,065,805						0.00		0.00						2,065,805	2,065,805	
		Supervision	1	LS	10,329,025						0.00		0.00						10,329,025	10,329,025	
		Temporary construction facilities	1	LS	5,164,512						0.00		0.00						5,164,512	5,164,512	
		Temporary utilities	1	LS	3,098,707						0.00		0.00						3,098,707	3,098,707	
		Safety	1	LS	5,164,512						0.00		0.00						5,164,512	5,164,512	
		Miscellaneous	1	LS	4,131,610						0.00		0.00						4,131,610	4,131,610	
		Subtotal Mobilizatior									$0		$0				$0	$0	$29,954,172	$29,954,172	
2		**Site Work**																			
02225		**Demolition**																			
02230		**Site Clearing**																			
SC	1	Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
SC	1	Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439	
EM	1	Scraper (Strip Peat Embankment Area)	915555	CY	1.03	B5b	0.006	5,493	274.7	17.17	94,298.47	0.00	0.00	1s1	2	154.87	850,764.54		0	945,063	
EM	1	Surface Prep. (Strip Peat Embankment Area)	1373332	SY	0.35	B5	0.008	10,987	915.6	17.22	189,226.83		0.00	3j4+	2	26	289,566.31		0	478,793	
EM	1 W/O	Scraper (Strip Peat Embankment Area)	364980	CY	1.03	B5b	0.006	2,190	109.5	17.17	37,591.48	0.00	0.00	1s1	2	154.87	339,151.84		0	376,743	
EM	1 W/O	Surface Prep. (Strip Peat Embankment Area)	547470	SY	0.35	B5	0.008	4,380	365.0	17.22	75,434.07		0.00	3j4+	2	26	115,433.76		0	190,868	
EM	1	Scraper (Strip Peat Inside Bench Area	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
EM	1 W/O	Scraper (Strip Peat Inside Bench Area	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504	
EM	1	Dozer Angle Blade (Strip Peat Inside Prod. Bl	485400	CY	1.15	B2	0.015	7,281	455	16.37	119,190	0.00	0.00	6c	1	120.02	436,945.59		0	556,135	
EM	1 W/O	Dozer Angle Blade (Strip Peat Inside Prod. Bl	15777	CY	1.15	B2	0.015	237	15	16.37	3,874	0.00	0.00	6c	1	120.02	14,202.41		0	18,077	$1.19
02240		**Dewatering**																			
		Seepage Canal																			
SC	2	Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507	
SC	2	12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
		Production Blast																			
SC	4 w/o	Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356	
SC	4 w/o	12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
		Cutoff Wall																			
SC	2 w/o	Pump,12" Suction (Make-up)	18803	LF	68.94	B4	0.040	752	94.0	16.87	12,686.65	0.00	0.00			68.27	1,283,700.00		0	1,296,387	
SC	2 w/o	12" Dia Pipe 2,500 GPM	54701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
02300		**Earthwork**																			
02305		**Equipment Mobilization Or Demobilizatior**																			
	12	Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049	Other: Driving Cost for 140Miles
	12	Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417	Other: Driving Cost for 140Miles
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12	Crawler Type Drill, 4'	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736	Other: Driving Cost for 140Miles
	12	Air Compressor, 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	Excavator, Diesel Hydraulic, Crawler Mounte	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	**Concrete Batch Plant, Portable, 200 CY/HR**	**8**	**EA**	**3,466**	**C1**	16.000	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane Over 75 Tor	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Rubber tired backhoe-loader, 3/4 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Wheeled Skid Steer, Diesel, w/ Broom	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Wash and Screen (Sand Horiz. Blanket)	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310		**Grading**																			
02315		**Excavation and Fil**			13																
SC	2	Drilling and Blasting (Seepage Canal Area	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2	Excavating Cap Rock (Seepage Canal Area	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2	Dump Truck (Canal/Stock Pile Area)	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2	Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2	Dozer Angle Blade (Stock Pile Area)	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
					13.32																***17,477,752***
SC	3	Excavated - Silty, Sand, Shells (Seepage Car	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3	Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812	
SC	3	Haul to Dewater and Work Stock Piles (Seepa	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719	(Avg. 1,000 FT. Around Trip)
SC	3	Dozer Angle Blade - Work Stock Piles (Seepa	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790	***3,608,106***
EM	4	Drilling and Blasting (Prod. Blast Area	2,912,398	CY	6.02	B6	0.080	232,992	9,708.0	17.11	3,987,267.07	1.89	5,504,432.28	6h	4	34.56	8,053,241.75		0	17,544,941	
EM	4	Excavating Cap Rock (Prod. Blast Area)	2,912,398	CY	1.75	B2	0.015	43,067	2,691.7	16.37	705,008.20	0.00	0.00	1b	1	102.03	4,394,268.88		0	5,099,277	
EM	4	Dump Truck (Prod. Blast Stock Pile Area)	3,640,498	CY	1.71	C1	0.017	60,328	7,541.0	11.88	716,699.55	0.00	0.00	3e1	1	91.31	5,508,804.50		0	6,225,504	(Avg. 2 Miles Around Trip)
EM	4	Hoe Ram (Stock Pile Area)	382,252	CY	1.74	A10	0.077	29,433	2,943.3	17.89	526,563.93	0.00	0.00	6i	1	5.91	139,208.32		0	665,772	
EM	4	Dozer Angle Blade (Prod. Blast Area)	3,640,498	CY	1.20	B2	0.016	57,338	3,583.6	16.37	938,620.38	0.00	0.00	6c	1	120.02	3,440,946.52		0	4,379,567	33,915,061
EM	4	Excavated - Silty, Sand, Shells (Prod. Blast Ar	4,732,360	CY	0.80	B3	0.005	23,662	985.9	16.75	396,335.17	0.00	0.00	1n	1	144.10	3,409,739.96		0	3,806,075	
EM	4	Haul to Dewater and Work Stock Piles (Silty, S	5,205,596	CY	1.71	C1	0.017	86,264	10,783.0	11.88	1,024,818.29	0.00	0.00	3e1	1	91.31	7,877,113.36		0	8,901,932	(Avg. 2 Miles Around Trip)
EM	4	Dozer Angle Blade (Work Stock Piles)	5,205,596	CY	0.69	B2	0.009	46,850	2,928.1	16.37	766,940.49	0.00	0.00	6c	1	120.02	2,811,574.59		0	3,578,515	16,286,522
																					52,481,516

EAA Reservoir A-1 Basis of Design Report

Case: 1b

January, 2006

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
2
07/26/05

EAA Reservoir A-1, Up Stream Rockfill Embankment (OG +26), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Dee
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

					Labor						Material		Equipment							
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	4 W/O Drilling and Blasting (Prod. Blast Area	94,664	CY	6.02	B6	0.080	7,573	315.5	17.11	129,601.45	1.89	178,915.13	6h	4	34.56	261,761.20		0	570,278	
EM	4 W/O Excavating Cap Rock (Prod. Blast Area)	94,664	CY	1.75	B2	0.015	1,400	87.5	16.37	22,915.47	0.00	0.00	1b	1	102.03	142,830.57		0	165,746	
EM	4 W/O Dozer Angle Blade (Prod. Blast Area)	113,597	CY	1.20	B2	0.016	1,789	111.8	16.37	29,288.41	0.00	0.00	6c	1	120.02	107,370.18		0	136,659	872,682
																				48,365,752
EM	4 W/O Excavated - Silty, Sand, Shells (Prod. Blast Ar	268,215	CY	0.80	B3	0.005	1,341	55.9	16.75	22,463.00	0.00	0.00	1n	1	144.10	193,253.06		0	215,716	
EM	4 W/O Haul to Dewater and Work Stock Piles (Silty, S	295,036	CY	1.71	C1	0.017	4,889	611.1	11.88	58,083.39	0.00	0.00	3e1	1	91.31	446,449.38		0	504,533	(Avg. 2 Miles Around Trip)
EM	4 W/O Dozer Angle Blade (Work Stock Piles)	295,036	CY	0.69	B2	0.009	2,655	166.0	16.37	43,467.71	0.00	0.00	6c	1	120.02	159,350.98		0	202,819	923,068
EM	5 Crusher (Level Coarse Borrow Area	126,034	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,287,510	2,287,510	
EM	5 Dozer Loader (Level Coarse Borrow Area)	126,034	CY	0.82	B5	0.013	1,607	133.9	17.22	27,676.66	0.00	0.00	6j	1	47.46	76,259.36		0	103,936	
EM	5 Dump Truck (Level Coarse Borrow Area)	176,447	CY	1.71	C1	0.017	2,924	365.5	11.88	34,736.88	0.00	0.00	3e1	1	91.31	266,999.84		0	301,737	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankment Area)	176,447	CY	1.03	B2	0.014	2,382	148.9	16.37	38,993.91	0.00	0.00	6c	1	120.02	142,950.20		0	181,944	2,875,127
EM	5 Cruncher (Transition Borrow Area	210,487.200	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,820,343	3,820,343	
EM	5 Dozer Loader (Transition Borrow Area	210,487.200	CY	0.82	B5	0.013	2,684	223.6	17.22	46,222.46	0.00	0.00	6j	1	47.46	127,359.83		0	173,582	
EM	5 Dump Truck (Transition Borrow Area	294,682.080	CY	1.71	C1	0.017	4,883	610.4	11.88	58,013.64	0.00	0.00	3e1	1	91.31	445,913.22		0	503,927	(Avg. 2 Miles Around Trip)
EM	5 Backhoe (Vertical New Embankment Area)	294,682.080	CY	3.28	B4a	0.153	45,086	939.3	17.08	770,150.14	0.00	0.00	2	1	4.33	195,186.36		0	965,337	5,463,188
EM	5 Crusher (Granular Toe Borrow Area	114,188	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,112,477	2,112,477	
EM	5 Dozer Loader (Granular Toe Borrow Area)	114,188	CY	0.82	B5	0.013	1,456	121.3	17.22	25,075.38	0.00	0.00	6j	1	47.46	69,091.87		0	94,167	
EM	5 Dump Truck (Granular Toe Borrow Area)	159,863	CY	1.71	C1	0.017	2,649	331.1	11.88	31,472.02	0.00	0.00	3e1	1	91.31	241,904.98		0	273,377	(Avg. 2 Miles Around Trip)
EM	5 Backhoe Loader (New Embankment Area)	159,863	CY	3.84	B4-	0.160	25,578	1,065.8	16.65	425,960.53	0.00	0.00	2a	1	7.33	187,524.25		0	613,485	
EM	5 Compacting (New Embankment Area)	159,863	CY	1.79	A1	0.089	14,228	1,778.5	17.51	249,129.04	0.00	0.00	8b2	1	2.64	37,595.51		0	286,725	3,380,230
																				8,843,418
EM	6 Clean Cap Rock Surface (Embankment Area)	163,419	SY	0.87	C1a	0.030	4,903	204.3	15.63	76,643.34		0.00	3j4	5	13.25	64,953.03		0	141,596	
EM	6 Cement Grout Cap Rock (Embankment Area)	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 W/O Clean Cap Rock Surface (Embankment Area)	267,960	SY	0.87	C1a	0.030	8,039	335.0	15.63	125,673.24		0.00	3j4	5	13.25	106,504.46		0	232,178	
EM	6 W/O Cement Grout Cap Rock (Embankment Area)	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 w/o Lean Concrete Fill In Cap Rock Cutoff Wall (E	91,168	CY	67.54	D8	0.137	12,490	260.2	17.69	220,885.93	65.00	5,925,920.00	8b1	2	0.83	10,376.69		0	6,157,183	
EM	6 w/o **Concrete Batch Plant and Delivery**	93,903	CY	26.15	C6a	0.480	45,073	704.3	14.44	650,635.38	0.00	0.00	8h1	6	40.04	1,804,643.90		0	2,455,279	
EM	6 w/o Cutoff Wall (Embankment Area)	2,051,280	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	5,128,200	0	5,128,200	(Soil Bentonite)
SC	6 w/o Cutoff Wall (Through Limestone)	15,043	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	676,922	0	676,922	14,791,358
EM	7 Dozer Loader (Sand Horiz. Filter)	198,290	CY	0.46	B2	0.006	1,190	74.4	16.37	19,476.08	0.00	0.00	6c	1	120.02	71,398.58		0	90,875	
EM	7 Dump Truck (Sand Horiz. Filter)	198,290	CY	0.93	C1	0.009	1,785	223.1	11.88	21,201.21	0.00	0.00	3e1	1	91.31	162,959.94		0	184,161	
EM	7 Wash and Screen (Sand Horiz. Filter)	198,290	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	779,281	0	779,281	
EM	7 Dozer Angle Blade (Sand Horiz. Filter	198,290	CY	0.69	B2	0.009	1,785	111.5	16.37	29,214.12	0.00	0.00	6c	1	120.02	107,097.87		0	136,312	
EM	7 Dozer and Dump Trucks (Sand Horiz. Filter	198,290	CY	1.75	C6	0.024	4,660	166.4	14.17	66,029.71	0.00	0.00	3f1	2	60.38	281,370.11		0	347,400	
EM	7 Dozer Angle Blade (Sand Horiz. Filter	198,290	CY	0.69	B2	0.009	1,785	111.5	16.37	29,214.12	0.00	0.00	6c	1	120.02	107,097.87		0	136,312	
EM	7 Compact (Sand Horiz. Filter)	198,290	CY	0.79	B5	0.009	1,785	148.7	17.22	30,736.99	0.00	0.00	6g1	1	70.54	125,889.60		0	156,627	
EM	7 Dozer Loader (Sand Vert. Filter)	136,752	CY	0.46	B2	0.006	821	51.3	16.37	13,431.78	0.00	0.00	6c	1	120.02	49,240.40		0	62,672	
EM	7 Dump Truck (Sand Vert. Filter)	136,752	CY	0.93	C1	0.009	1,231	153.8	11.88	14,621.52	0.00	0.00	3e1	1	91.31	112,386.17		0	127,008	
EM	7 Wash and Screen (Sand Vert. Filter)	136,752	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	537,435	0	537,435	
EM	7 Dozer Angle Blade (Sand Vert. Filter)	136,752	CY	0.69	B2	0.009	1,231	76.9	16.37	20,147.67	0.00	0.00	6c	1	120.02	73,860.60		0	94,008	
EM	7 Dozer and Dump Trucks (Sand Vert. Filter	136,752	CY	1.75	C6	0.024	3,214	114.8	14.17	45,537.73	0.00	0.00	3f1	2	60.38	194,048.35		0	239,586	
EM	7 Backhoe (Sand Vert. Filter)	136,752	CY	3.28	B4a	0.153	20,923	435.9	17.08	357,400.67	0.00	0.00	2	1	4.33	90,579.40		0	447,980	
EM	8 Dozer and Dump Trucks (Max. 6" Random Fil	1,490,597	CY	1.75	C6	0.024	35,029	1,251.0	14.17	496,361.28	0.00	0.00	3f1	2	60.38	2,115,127.03		0	2,611,488	
EM	8 Dozer With Ripper Attach, (Max. 6" Random F	1,490,597	CY	2.04	B5	0.020	29,812	2,484.3	17.22	513,460.91	0.00	0.00	6g	2	84.81	2,528,406.98		0	3,041,868	
EM	8 Dozer Angle Blade (Max. 6" Random Fill	1,490,597	CY	0.69	B2	0.009	13,415	838.5	16.37	219,609.63	0.00	0.00	6c	1	120.02	805,080.52		0	1,024,690	
EM	8 Compact (Max. 6" Random Fill)	1,490,597	CY	0.79	B5	0.009	13,415	1,117.9	17.22	231,057.41	0.00	0.00	6g1	1	70.54	946,342.51		0	1,177,400	
EM	8 W/O Dozer and Dump Trucks (Max. 6" Random Fil	1,496,880	CY	1.75	C6	0.024	35,177	1,256.3	14.17	498,453.56	0.00	0.00	3f1	2	60.38	2,124,042.76		0	2,622,496	
EM	8 W/O Dozer With Ripper Attach, (Max. 6" Random F	1,496,880	CY	2.04	B5	0.020	29,938	2,494.8	17.22	515,625.26	0.00	0.00	6g	2	84.81	2,539,064.79		0	3,054,690	
EM	8 W/O Dozer Angle Blade (Max. 6" Random Fill	1,496,880	CY	0.69	B2	0.009	13,472	842.0	16.37	220,535.33	0.00	0.00	6c	1	120.02	808,474.11		0	1,029,009	
EM	8 W/O Compact (Max. 6" Random Fill)	1,496,880	CY	0.79	B5	0.009	13,472	1,122.7	17.22	232,031.37	0.00	0.00	6g1	1	70.54	950,331.56		0	1,182,363	
EM	8 Dozer and Dump Trucks (Mass Random Fill	2,062,904	CY	1.75	C6	0.024	48,478	1,731.4	14.17	686,936.69	0.00	0.00	3f1	2	60.38	2,927,219.38		0	3,614,156	
EM	8 Dozer Angle Blade (Mass Random Fill	2,062,904	CY	0.69	B2	0.009	18,566	1,160.4	16.37	303,927.63	0.00	0.00	6c	1	120.02	1,114,187.12		0	1,418,115	
EM	8 Compact (Mass Random Fill)	2,062,904	CY	0.79	B5	0.009	18,566	1,547.2	17.22	319,770.74	0.00	0.00	6g1	1	70.54	1,309,685.95		0	1,629,457	22,405,733
EM	8 W/O Dozer and Dump Trucks (Mass Random Fill	1,071,840	CY	1.75	C6	0.024	25,188	899.6	14.17	356,917.36	0.00	0.00	3f1	2	60.38	1,520,919.51		0	1,877,837	
EM	8 W/O Dozer Angle Blade (Mass Random Fill	1,071,840	CY	0.69	B2	0.009	9,647	602.9	16.37	157,914.19	0.00	0.00	6c	1	120.02	578,907.39		0	736,822	
EM	8 W/O Compact (Mass Random Fill)	1,071,840	CY	0.79	B5	0.009	9,647	803.9	17.22	166,145.92	0.00	0.00	6g1	1	70.54	680,484.33		0	846,630	25,867,021
EM	8 Dozer Loader (Rock Fill Borrow Area)	2,602,908	CY	1.15	B2	0.015	39,044	2,440.2	16.37	639,144.06	0.00	0.00	6c	1	120.02	2,343,077.75		0	2,982,222	
EM	8 Dump Truck (Rock Fill Borrow Area)	3,644,071	CY	0.93	C1	0.009	32,797	4,099.6	11.88	389,624.09	0.00	0.00	3e1	1	91.31	2,994,787.64		0	3,384,412	
EM	8 Dozer Angle Blade (New Embankment Area)	3,644,071	CY	1.03	B2	0.014	49,195	3,074.7	16.37	805,321.51	0.00	0.00	6c	1	120.02	2,952,277.97		0	3,757,599	10,124,233

Case: 1b

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
2
07/26/05

EAA Reservoir A-1, Up Stream Rockfill Embankment (OG +26), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Dee
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	10 Common Borrow (Top Soil - Peat)	225,641	CY	3.65	C6	0.047	10,605	378.8	14.17	150,274.52	0.00	0.00	3f	3	63.57	674,182.48		0	824,457	
EM	10 W/O Common Borrow (Top Soil - Peat)	97,020	CY	3.65	C6	0.047	4,560	162.9	14.17	64,614.35	0.00	0.00	3f	3	63.57	289,881.90		0	354,496	1,178,953
02370	**Erosion and Sedimentation Contro**																			
02600	**Drainage and Containmen**																			
02620	**Subdrainage System (Seepage Water)**																			
02700	**Bases, Ballasts, Pavements and Appurtenance:**																			
EM	8 w/o Crusher (Transition Borrow Area	74177	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		927,217	927,217	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" Thick	222532	SY	1.13	B7a	0.019	4,256	59.1	16.28	69,281.74	0.00	0.00	7a+	7	42.93	182,704.54		0	251,986	1,179,203
02910	**Plant Preparation**																			
EM	10 Fine Grading	670,085	SY	0.32	B5a	0.008	5,361	335.0	17.30	92,712.93	0.00	0.00	7d	1	22.97	123,136.33		0	215,849	
EM	10 W/O Fine Grading	291,060	SY	0.32	B5a	0.008	2,328	145.5	17.30	40,271.06	0.00	0.00	7d	1	22.97	53,485.86		0	93,757	
02920	**Lawns and Grasses**																			
EM	10 Hydro or Air Seeding w/ Mulch and Fertilize	670,085	SY	0.24	C4a	0.003	2,010	3.0	15.49	31,138.84	0.15	100,512.72	3j5	2	14.11	28,360.99		0	160,013	469,619
EM	10 W/O Hydro or Air Seeding w/ Mulch and Fertilize	291,060	SY	0.24	C4a	0.003	873	3.0	15.49	13,525.56	0.15	43,659.00	3j5	2	14.11	12,318.96		0	69,504	539,122
	Subtotal Site Constructior						1,562,823			$24,644,842		$14,050,873				$96,132,282	$7,121,839	$9,250,795	$151,200,631	
3	**Concrete**																			
03050	**Basic Concrete Materials and Methods**																			
03100	**Concrete and Forms and Accessories**																			
03200	**Concrete Reinforcement**																			
03300	**Cast-In-Place Concrete**																			
03310	**Structural C+C270oncrete**																			
	Roller Compacted Concrete																			
EM	11 Mass Placement, 1' Lift, 12" Layer	205,128	CY	1.07	B5	0.009	1,846	76.9	16.91	31,212.28		0.00	6f1	2	102.16	188,599.39		0	219,812	
EM	11 Sloped Face, Nonformed, 1' Lift	218,803	CY	5.00	B5	0.042	9,190	382.9	16.91	155,367.78		0.00	6f1	2	102.16	938,805.85		0	1,094,174	
EM	11 Roller Compacted Concrete, 1.5"-2" Agg., 20(	423,931	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	19,076,904.00				0.00		0	19,076,904	100 lbs of cement by volume
EM	11 Dump Truck (18 CY) Conveying Materia	423,931	CY	1.24	C1	0.012	5,087	635.9	11.88	60,435.63		0.00	3e1	1	91.31	464,529.50		0	524,965	(15 Min. Cycles)
EM	11 Truck Mtd. Hydraulic Crane 100 Ton, Conc. B	205,128	CY	0.59	D18a	0.025	5,128	128.2	17.43	89,405.04		0.00	8g	4	6.32	32,389.38		0	121,794	$49.63
EM	11 Surface Prep. Vacuum Truck	*307,692*	SY	0.18	C6a-	0.006	1,846	92.3	17.72	32,710.12		0.00	3j2	1	12.86	23,734.74		0	56,445	
EM	11 Surface Prep. Water Clean	307,692	SY	0.22	C1a	0.008	2,462	102.6	15.63	38,482.01		0.00	3j3	4	11.43	28,138.83		0	66,621	
EM	11 Surface Prep. Water Blast	307,692	SY	0.87	C1a	0.030	9,231	384.6	15.63	144,307.55		0.00	3j4	5	13.25	122,296.50		0	266,604	*$50.54*
EM	**11 Concrete Batch Plant and Delivery**	**436,649**	**CY**	**26.15**	**C6a**	**0.480**	**209,592**	**3,274.9**	**14.44**	**3,025,454.53**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**8,391,594.14**		**0**	**11,417,049**	$77.48
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	138,600	CY	1.07	B5	0.009	1,247	52.0	16.91	21,089.38		0.00	6f1	2	102.16	127,432.02		0	148,521	
EM	11 W/O Vertical Face, Formed, 1' Lift	147,840	CY	7.14	B5	0.060	8,870	369.6	16.91	149,968.90		0.00	6f1	2	102.16	906,183.25		0	1,056,152	
EM	11 W/O Roller Compacted Concrete, 1.5"-2" Agg., 20(	286,440	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	12,889,800.00				0.00		0	12,889,800	100 lbs of cement by volume
EM	11 W/O Dump Truck (18 CY) Conveying Materia	286,440	CY	1.24	C1	0.012	3,437	429.7	11.88	40,834.89		0.00	3e1	1	91.31	313,871.28		0	354,706	(15 Min. Cycles)
EM	11 W/O Truck Mtd. Hydraulic Crane 100 Ton, Conc. B	286,440	CY	0.44	D18a	0.025	7,161	179.0	17.43	124,844.87		0.00	8g	0	0.00	0.00		0	124,845	
EM	11 W/O Surface Prep. Vacuum Truck	*207,900*	SY	0.11	C6a-	0.006	1,247	62.4	17.72	22,101.43		0.00	3j2	0	0.00	0.00		0	22,101	
EM	11 W/O Surface Prep. Water Clean	207,900	SY	0.22	C1a	0.008	1,663	69.3	15.63	26,001.36		0.00	3j3	4	11.43	19,012.72		0	45,014	
EM	11 W/O Surface Prep. Water Blast	207,900	SY	0.87	C1a	0.030	6,237	259.9	15.63	97,505.10		0.00	3j4	5	13.25	82,632.77		0	180,138	$51.74
EM	**11 W/O Concrete Batch Plant and Delivery**	**295,033**	**CY**	**26.15**	**C6a**	**0.480**	**141,616**	**2,212.7**	**14.44**	**2,044,226.04**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**5,669,996.04**		**0**	**7,714,222**	$50.88
																				$78.67
03400	**Precast Concrete**																			
03500	**Cementitious Decks and Underlay**																			
03600	**Grouts**																			
03900	**Concrete Restorations and Cleaning**																			
		0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0	*55,379,867*
	Subtotal Concrete						415,861			$6,103,947		$31,966,704				$17,309,216	$0	$0	$55,379,867	
	Construction Subtotal (Direct Costs)						1,978,684			$30,748,789		$46,017,577				$113,441,498	$7,121,839	$39,204,968	**$236,534,671**	

Indirect Costs			
Sales Tax	6%	of purchased materials + Rental Equipment	9,567,544
Overhead and Profit	17%	of construction cost + general requirements	39,534,552
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	9,997,287
Project Reserve	5%	of construction cost	14,781,703
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	93,124,727
Construction Subtotal Indirects			**$167,005,812**
Total Construction (Direct and Indirect Costs			**$403,540,483**
Permits			0
Design	0%	of construction cost	0
Construction Managemen	0%	of construction cost	0
Total			**$403,540,483**

TABLE 8.13-2

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 2

Case: 2

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02

Project No.: 141731.0910

Revision No.: 2

Date: 07/26/05

Opinion Of Probable Cost

EAA Reservoir A-1, Embankment (OG +26) "Geomembrane," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

South Side Embankment and Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				59,967	1,025,255	0	6,263,708	0	0	7,288,963	0.9422	6,867,790	14,156,753
	Subtotal				59,967	$1,025,255	$0	$6,263,708	$0	$0	$7,288,963		$6,867,790	$14,156,753
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				187,788	3,085,457	2,297,434	13,173,450	0	0	18,556,341	0.9422	17,484,116	36,040,457
	Subtotal				187,788	$3,085,457	$2,297,434	$13,173,450	$0	$0	$18,556,341		$17,484,116	$36,040,457
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.9422	3,399,622	7,007,728
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,399,622	$7,007,728
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				533,492	8,348,548	4,950,830	36,030,297	0	0	49,329,675	0.9422	46,479,299	95,808,974
	Subtotal				533,492	$8,348,548	$4,950,830	$36,030,297	$0	$0	$49,329,675		$46,479,299	$95,808,974
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				95,323	1,519,851	0	4,094,267	0	12,240,269	17,854,386	0.9422	16,822,721	34,677,107
	Subtotal				95,323	$1,519,851	$0	$4,094,267	$0	$12,240,269	$17,854,386		$16,822,721	$34,677,107
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				73,650	1,123,006	5,925,920	2,028,147	9,727,502	0	18,804,576	0.9422	17,718,007	36,522,582
	Subtotal				73,650	$1,123,006	$5,925,920	$2,028,147	$9,727,502	$0	$18,804,576		$17,718,007	$36,522,582
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				64,991	980,128	0	4,282,420	3,899,457	0	9,162,004	0.9422	8,632,604	17,794,608
	Subtotal				64,991	$980,128	$0	$4,282,420	$3,899,457	$0	$9,162,004		$8,632,604	$17,794,608
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				474,734	7,602,914	11,888,520	15,994,908	0	822,189	36,308,531	0.9422	34,210,545	70,519,076
	Subtotal				474,734	$7,602,914	$11,888,520	$15,994,908	$0	$822,189	$36,308,531		$34,210,545	$70,519,076
9	**Embankment Construction (Broken Cap Rock And Silty Sand, Shell, Etc. Soils)**													
	Materials and Methods				141,839	2,171,449	0	4,678,216	0	0	6,849,665	0.9422	6,453,876	13,303,541
	Subtotal				141,839	$2,171,449	$0	$4,678,216	$0	$0	$6,849,665		$6,453,876	$13,303,541
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				29,869	455,659	167,911	1,369,101	0	0	1,992,671	0.9422	1,877,530	3,870,202
	Subtotal				29,869	$455,659	$167,911	$1,369,101	$0	$0	$1,992,671		$1,877,530	$3,870,202
11	**Embankment Construction (Cutoff Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				436,018	6,401,480	33,358,802	17,901,111	0	0	57,661,393	0.9422	54,329,592	111,990,986
	Subtotal				436,018	$6,401,480	$33,358,802	$17,901,111	$0	$0	$57,661,393		$54,329,592	$111,990,986
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.9422	157,857	325,395
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$157,857	$325,395
	Total				**2,132,054**	**$33,207,607**	**$58,589,417**	**$108,994,160**	**$13,626,959**	**$13,165,707**	**$227,583,850**		**$214,433,560**	**$442,017,410**

Case: 2

January 2006

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02

Project No. 141731.0910

Revision No.: 2

Date: 07/26/05

Opinion Of Probable Cost

EAA Reservoir A-1, Embankment (OG +26) "Geomembrane," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

					Labor						Material		Equipment							
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1	**General Requirements**																			
	Mobilization	1	LS	2,275,839						0.00		0.00						2,275,839	2,275,839	
	Supervision	1	LS	11,379,193						0.00		0.00						11,379,193	11,379,193	
	Temporary Construction Facilities	1	LS	5,689,596						0.00		0.00						5,689,596	5,689,596	
	Temporary Utilities	1	LS	3,413,758						0.00		0.00						3,413,758	3,413,758	
	Safety	1	LS	5,689,596						0.00		0.00						5,689,596	5,689,596	
	Miscellaneous	1	LS	4,551,677						0.00		0.00						4,551,677	4,551,677	
	Subtotal Mobilization									$0		$0				$0	$0	$32,999,658	$32,999,658	
2	**Site Work**																			
02225	**Demolition**																			
02230	**Site Clearing**																			
SC	1 Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
SC	1 Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439	
EM	1 Scraper (Strip Peat Embankment Area)	915555	CY	1.03	B5b	0.006	5,493	274.7	17.17	94,298.47	0.00	0.00	1s1	2	154.87	850,764.54		0	945,063	
EM	1 Surface Prep. (Strip Peat Embankment A	1373332	SY	0.35	B5	0.008	10,987	915.6	17.22	189,226.83		0.00	3j4+	2	26	289,566.31		0	478,793	
EM	1 W/O Scraper (Strip Peat Embankment Area)	642180	CY	1.03	B5b	0.006	3,853	192.7	17.17	66,141.97	0.00	0.00	1s1	2	154.87	596,735.52		0	662,877	
EM	1 W/O Surface Prep. (Strip Peat Embankment A	963270	SY	0.35	B5	0.008	7,706	642.2	17.22	132,725.76		0.00	3j4+	2	26	203,104.96		0	335,831	
EM	1 Scraper (Strip Peat Inside Bench Area)	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
EM	1 W/O Scraper (Strip Peat Inside Bench Area)	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504	
EM	1 Dozer Angle Blade (Strip Peat Inside Pro	105490	CY	1.15	B2	0.015	1,582	99	16.37	25,903	0.00	0.00	6c	1	120.02	94,959.92		0	120,863	
EM	1 W/O Dozer Angle Blade (Strip Peat Inside Pro	331091	CY	1.15	B2	0.015	4,966	310	16.37	81,299	0.00	0.00	6c	1	120.02	298,040.33		0	379,340	$1.20
02240	**Dewatering**																			
	Seepage Canal																			
SC	2 Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507	
SC	2 12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
	Production Blast																			
SC	4 w/o Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356	
SC	4 w/o 12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
	Cutoff Wall																			
SC	2 w/o Pump,12" Suction (Make-up)	31508	LF	68.94	B4	0.040	1,260	157.5	16.87	21,258.71	0.00	0.00			68.27	2,151,064.86		0	2,172,324	
SC	2 w/o 12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
02300	**Earthwork**																			
02305	**Equipment Mobilization Or Demobilization**																			
	12 Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049	Other: Driving Cost for 140Miles
	12 Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417	Other: Driving Cost for 140Miles
	12 Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12 Crawler Type Drill, 4"	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736	Other: Driving Cost for 140Miles
	12 Air Compressor., 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 Excavator, Diesel Hydraulic, Crawler Mo	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12 Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Concrete Batch Plant, Portable, 200 C	**8**	**EA**	**3,466**	**C1**	**16.000**	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12 Concrete Transit Mixer Truck	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12 Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12 Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12 Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12 Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Rubber tired backhoe-loader, 3/4 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Wheeled Skid Steer, Diesel, w/ Broom	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12 Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12 Wash and Screen (Sand Horiz. Blanket)	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310	**Grading**																			
02315	**Excavation and Fill**			13																
SC	2 Drilling and Blasting (Seepage Canal Are	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2 Excavating Cap Rock (Seepage Canal A	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2 Dump Truck (Canal/Stock Pile Area)	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2 Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2 Dozer Angle Blade (Stock Pile Area)	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
				13.32																***18,556,341***
SC	3 Excavated - Silty, Sand, Shells (Seepage	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3 Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812	
SC	3 Haul to Dewater and Work Stock Piles (S	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719	(Avg. 1,000 FT. Around Trip)
SC	3 Dozer Angle Blade - Work Stock Piles (S	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790	***3,608,106***
EM	4 Drilling and Blasting (Prod. Blast Area)	632,942	CY	6.02	B6	0.080	50,635	2,109.8	17.11	866,539.34	1.89	1,196,259.75	6h	4	34.56	1,750,183.94		0	3,812,983	
EM	4 Excavating Cap Rock (Prod. Blast Area)	632,942	CY	1.75	B2	0.015	9,360	585.0	16.37	153,217.06	0.00	0.00	1b	1	102.03	954,991.68		0	1,108,209	
EM	4 Dump Truck (Prod. Blast Stock Pile Area	791,177	CY	1.71	C1	0.017	13,111	1,638.9	11.88	155,757.90	0.00	0.00	3e1	1	91.31	1,197,209.95		0	1,352,968	(Avg. 2 Miles Around Trip)
EM	4 Hoe Ram (Stock Pile Area)	83,074	CY	1.74	A10	0.077	6,397	639.7	17.89	114,436.37	0.00	0.00	6i	1	5.91	30,253.68		0	144,690	

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/26/05

EAA Reservoir A-1, Embankment (OG +26) "Geomembrane," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95	Miles	=	68,376 Total Linear Feet
Without Seepage Control (South Side)	8.75	Miles	=	46,200 Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	4 Dozer Angle Blade (Prod. Blast Area)	791,177	CY	1.20	B2	0.016	12,461	778.8	16.37	203,987.21	0.00	0.00	6c	1	120.02	747,809.34		0	951,797	7,370,646
EM	4 Excavated - Silty, Sand, Shells (Prod. Bla	579,662	CY	0.80	B3	0.005	2,898	120.8	16.75	48,546.72	0.00	0.00	1n	1	144.10	417,655.84		0	466,203	
EM	4 Haul to Dewater and Work Stock Piles (S	637,629	CY	1.71	C1	0.017	10,566	1,320.8	11.88	125,529.03	0.00	0.00	3e1	1	91.31	964,860.21		0	1,090,389	(Avg. 2 Miles Around Trip)
EM	4 Dozer Angle Blade (Work Stock Piles)	637,629	CY	0.69	B2	0.009	5,739	358.7	16.37	93,941.82	0.00	0.00	6c	1	120.02	344,387.13		0	438,329	1,994,921
																				11,645,500
EM	4 W/O Drilling and Blasting (Prod. Blast Area)	1,986,545	CY	6.02	B6	0.080	158,924	6,621.8	17.11	2,719,712.56	1.89	3,754,570.07	6h	4	34.56	5,493,111.53		0	11,967,394	
EM	4 W/O Excavating Cap Rock (Prod. Blast Area)	1,986,545	CY	1.75	B2	0.015	29,376	1,836.0	16.37	480,885.68	0.00	0.00	1b	1	102.03	2,997,328.26		0	3,478,214	
EM	4 W/O Dozer Angle Blade (Prod. Blast Area)	2,383,854	CY	1.20	B2	0.016	37,546	2,346.6	16.37	614,623.12	0.00	0.00	6c	1	120.02	2,253,184.93		0	2,867,808	18,313,416
																				39,262,071
EM	4 W/O Excavated - Silty, Sand, Shells (Prod. Bla	5,628,544	CY	0.80	B3	0.005	28,143	1,172.6	16.75	471,390.58	0.00	0.00	1n	1	144.10	4,055,454.65		0	4,526,845	
EM	4 W/O Haul to Dewater and Work Stock Piles (S	6,191,399	CY	1.71	C1	0.017	102,600	12,825.0	11.88	1,218,891.80	0.00	0.00	3e1	1	91.31	9,368,830.57		0	10,587,722	(Avg. 2 Miles Around Trip)
EM	4 W/O Dozer Angle Blade (Work Stock Piles)	6,191,399	CY	0.69	B2	0.009	55,723	3,482.7	16.37	912,178.76	0.00	0.00	6c	1	120.02	3,344,012.56		0	4,256,191	19,370,759
EM	5 Crusher (Transition Borrow Area)	379,247	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		6,883,325	6,883,325	
EM	5 Dozer Loader (Transition Borrow Area)	379,247	CY	0.82	B5	0.013	4,835	402.9	17.22	83,281.60	0.00	0.00	6j	1	47.46	229,471.33		0	312,753	
EM	5 Dump Truck (Transition Borrow Area)	530,945	CY	1.71	C1	0.017	8,799	1,099.8	11.88	104,526.42	0.00	0.00	3e1	1	91.31	803,426.78		0	907,953	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankment A	530,945	CY	1.03	B2	0.014	7,168	448.0	16.37	117,336.23	0.00	0.00	6c	1	120.02	430,150.15		0	547,486	8,651,518
EM	5 Cruncher (Transition Borrow Area)	17,186	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		311,933	311,933	
EM	5 Dozer Loader (Transition Borrow Area)	17,186	CY	0.82	B5	0.013	219	18.3	17.22	3,774.09	0.00	0.00	6j	1	47.46	10,399.00		0	14,173	
EM	5 Dump Truck (Transition Borrow Area)	24,061	CY	1.71	C1	0.017	399	49.8	11.88	4,736.85	0.00	0.00	3e1	1	91.31	36,409.07		0	41,146	(Avg. 2 Miles Around Trip)
EM	5 Backhoe (Vertical New Embankment Are	24,061	CY	3.28	B4a	0.153	3,681	76.7	17.08	62,883.20	0.00	0.00	2	1	4.33	15,937.08		0	78,820	446,072
EM	5 Cruncher (Transition Borrow Area)	181,030	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,285,696	3,285,696	
EM	5 Dozer Loader (Transition Borrow Area)	181,030	CY	0.82	B5	0.013	2,308	192.3	17.22	39,753.75	0.00	0.00	6j	1	47.46	109,536.16		0	149,290	
EM	5 Dump Truck (Transition Borrow Area)	253,442	CY	1.71	C1	0.017	4,200	525.0	11.88	49,894.79	0.00	0.00	3e1	1	91.31	383,508.86		0	433,404	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (Sloped New Embank	253,442	CY	8.18	B2	0.107	27,144	1,696.5	16.37	444,341.55	0.00	0.00	6c	1	120.02	1,628,939.19		0	2,073,281	5,941,670
EM	5 Crusher (Granular Toe Borrow Area)	95,098	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,759,314	1,759,314	
EM	5 Dozer Loader (Granular Toe Borrow Area	95,098	CY	0.82	B5	0.013	1,213	101.0	17.22	20,883.30	0.00	0.00	6j	1	47.46	57,541.15		0	78,424	
EM	5 Dump Truck (Granular Toe Borrow Area)	133,137	CY	1.71	C1	0.017	2,206	275.8	11.88	26,210.55	0.00	0.00	3e1	1	91.31	201,463.51		0	227,674	(Avg. 2 Miles Around Trip)
EM	5 Backhoe Loader (New Embankment Area	133,137	CY	3.84	B4-	0.160	21,302	887.6	16.65	354,748.81	0.00	0.00	2a	1	7.33	156,174.10		0	510,923	
EM	5 Compacting (New Embankment Area)	133,137	CY	1.79	A1	0.089	11,849	1,481.2	17.51	207,479.86	0.00	0.00	8b2	1	2.64	31,310.33		0	238,790	2,815,126
																				17,854,386
EM	6 Clean Cap Rock Surface (Embankment A	320,000	SY	0.87	C1a	0.030	9,600	400.0	15.63	150,079.85		0.00	3j4	5	13.25	127,188.36		0	277,268	
EM	6 Cement Grout Cap Rock (Embankment A	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 W/O Clean Cap Rock Surface (Embankment A	216,216	SY	0.87	C1a	0.030	6,486	270.3	15.63	101,405.30		0.00	3j4	5	13.25	85,938.08		0	187,343	
EM	6 W/O Cement Grout Cap Rock (Embankment A	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 w/o Lean Concrete Fill In Cap Rock Cutoff W	91,168	CY	67.54	D8	0.137	12,490	260.2	17.69	220,885.93	65.00	5,925,920.00	8b1	2	0.83	10,376.69		0	6,157,183	
EM	**6 w/o Concrete Batch Plant and Delivery**	**93,903**	**CY**	**26.15**	**C6a**	**0.480**	**45,073**	**704.3**	**14.44**	**650,635.38**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,804,643.90**		**0**	**2,455,279**	
EM	6 w/o Cutoff Wall (Embankment Area)	3,437,280	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	8,593,200	0	8,593,200	(Soil Bentonite)
SC	6 w/o Cutoff Wall (Through Limestone)	25,207	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	1,134,302	0	1,134,302	18,804,576
EM	9 Dozer and Dump Trucks (Select Fill Belo	3,417,802	CY	0.74	C6	0.024	80,318	2,868.5	14.17	1,138,111.00	0.00	0.00	3f1	2	60.38	1,385,655.07		0	2,523,766	
EM	9 Dozer Angle Blade (Select Fill Below Dra	3,417,802	CY	0.69	B2	0.009	30,760	1,922.5	16.37	503,544.78	0.00	0.00	6c	1	120.02	1,845,975.97		0	2,349,521	
EM	9 Compact (Select Fill Below Drain)	3,417,802	CY	0.58	B5	0.009	30,760	2,563.4	17.22	529,793.50	0.00	0.00	6g1	1	70.54	1,446,584.53		0	1,976,378	6,849,665
EM	7 Dozer Loader (Sand Horiz. Filter)	261,233	CY	0.46	B2	0.006	1,567	98.0	16.37	25,658.33	0.00	0.00	6c	1	120.02	94,062.47		0	119,721	
EM	7 Dump Truck (Sand Horiz. Filter)	261,233	CY	0.93	C1	0.009	2,351	293.9	11.88	27,931.06	0.00	0.00	3e1	1	91.31	214,687.96		0	242,619	
EM	7 Wash and Screen (Sand Horiz. Filter)	261,233	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	1,026,647	0	1,026,647	
EM	7 Dozer Angle Blade (Sand Horiz. Filter)	261,233	CY	0.69	B2	0.009	2,351	146.9	16.37	38,487.50	0.00	0.00	6c	1	120.02	141,093.70		0	179,581	
EM	7 Dozer and Dump Trucks (Sand Horiz. Fil	261,233	CY	1.75	C6	0.024	6,139	219.2	14.17	86,989.38	0.00	0.00	3f1	2	60.38	370,684.80		0	457,674	
EM	7 Dozer Angle Blade (Sand Horiz. Filter)	261,233	CY	0.69	B2	0.009	2,351	146.9	16.37	38,487.50	0.00	0.00	6c	1	120.02	141,093.70		0	179,581	
EM	7 Compact (Sand Horiz. Filter)	261,233	CY	0.79	B5	0.009	2,351	195.9	17.22	40,493.77	0.00	0.00	6g1	1	70.54	165,850.46		0	206,344	
EM	7 Dozer Loader (Sand Horiz. Drain)	730,995	CY	0.46	B2	0.006	4,386	274.1	16.37	71,798.32	0.00	0.00	6c	1	120.02	263,209.89		0	335,008	
EM	7 Dump Truck (Sand Horiz. Drain)	730,995	CY	0.93	C1	0.009	6,579	822.4	11.88	78,157.97	0.00	0.00	3e1	1	91.31	600,749.63		0	678,908	
EM	7 Wash and Screen (Sand Horiz. Drain)	730,995	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	2,872,810	0	2,872,810	
EM	7 Dozer Angle Blade (Sand Horiz. Drain)	730,995	CY	0.69	B2	0.009	6,579	411.2	16.37	107,697.48	0.00	0.00	6c	1	120.02	394,814.84		0	502,512	
EM	7 Dozer and Dump Trucks (Sand Horiz. Dr	730,995	CY	1.75	C6	0.024	17,178	613.5	14.17	243,417.64	0.00	0.00	3f1	2	60.38	1,037,267.11		0	1,280,685	
EM	7 Dozer Angle Blade (Sand Horiz. Drain)	730,995	CY	0.69	B2	0.009	6,579	411.2	16.37	107,697.48	0.00	0.00	6c	1	120.02	394,814.84		0	502,512	
EM	7 Compact (Sand Horiz. Drain)	730,995	CY	0.79	B5	0.009	6,579	548.2	17.22	113,311.52	0.00	0.00	6g1	1	70.54	464,090.31		0	577,402	
EM	8 Dozer and Dump Trucks (Mass Random	3,226,663	CY	1.75	C6	0.024	75,827	2,708.1	14.17	1,074,462.79	0.00	0.00	3f1	2	60.38	4,578,570.85		0	5,653,034	
EM	8 Dozer Angle Blade (Mass Random Fill)	3,226,663	CY	0.69	B2	0.009	29,040	1,815.0	16.37	475,384.32	0.00	0.00	6c	1	120.02	1,742,740.81		0	2,218,125	
EM	8 Compact (Mass Random Fill)	3,226,663	CY	0.79	B5	0.009	29,040	2,420.0	17.22	500,165.10	0.00	0.00	6g1	1	70.54	2,048,527.67		0	2,548,693	10,419,852
EM	8 W/O Dozer and Dump Trucks (Mass Random	2,180,178	CY	1.75	C6	0.024	51,234	1,829.8	14.17	725,988.37	0.00	0.00	3f1	2	60.38	3,093,628.95		0	3,819,617	
EM	8 W/O Dozer Angle Blade (Mass Random Fill)	2,180,178	CY	0.69	B2	0.009	19,622	1,226.4	16.37	321,205.62	0.00	0.00	6c	1	120.02	1,177,527.57		0	1,498,733	
EM	8 W/O Compact (Mass Random Fill)	2,180,178	CY	0.79	B5	0.009	19,622	1,635.1	17.22	337,949.39	0.00	0.00	6g1	1	70.54	1,384,140.32		0	1,722,090	17,460,292
EM	10 Common Borrow (Top Soil - Peat)	222,906	CY	3.65	C6	0.047	10,477	374.2	14.17	148,453.01	0.00	0.00	3f	3	63.57	666,010.57		0	814,464	
EM	10 W/O Common Borrow (Top Soil - Peat)	150,612	CY	3.65	C6	0.047	7,079	252.8	14.17	100,306.09	0.00	0.00	3f	3	63.57	450,007.14		0	550,313	1,364,777

Case: 2

January 2006

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/26/05

EAA Reservoir A-1, Embankment (OG +26) "Geomembrane," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
02370	**Erosion and Sedimentation Control**																			
02600	**Drainage and Containment**																			
02620	**Subdrainage System (Seepage Water)**																			
EM	8 Geomembrane Placement	978,479	SY	18.19	B4-	0.252	246,577	10,274.0	16.65	4,106,324.28	12.15	11,888,520.34	2a	1	7.33	1,807,762.28		0	17,802,607	
02700	**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o Crusher (Transition Borrow Area)	65,775	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		822,189	822,189	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" Thick	197,325	SY	1.13	B7a	0.019	3,774	52.4	16.28	61,434.04	0.00	0.00	7a+	7	42.93	162,009.17		0	223,443	18,848,239
02910	**Plant Preparation**																			
EM	10 Fine Grading	668,034	SY	0.32	B5a	0.008	5,344	334.0	17.30	92,429.12	0.00	0.00	7d	1	22.97	122,759.39		0	215,189	
EM	10 W/O Fine Grading	451,374	SY	0.32	B5a	0.008	3,611	225.7	17.30	62,452.11	0.00	0.00	7d	1	22.97	82,945.53		0	145,398	
02920	**Lawns and Grasses**																			
EM	10 Hydro or Air Seeding w/ Mulch and Fertil	668,034	SY	0.24	C4a	0.003	2,004	3.0	15.49	31,043.52	0.15	100,205.03	3j5	2	14.11	28,274.17		0	159,523	520,109
EM	10 W/O Hydro or Air Seeding w/ Mulch and Fertil	451,374	SY	0.24	C4a	0.003	1,354	3.0	15.49	20,975.35	0.15	67,706.10	3j5	2	14.11	19,104.17		0	107,786	627,894
	Subtotal Site Construction						1,696,036			$26,806,128		$25,230,615				$91,093,048	$13,626,959	$13,165,707	$169,922,457	
3	**Concrete**																			
03050	**Basic Concrete Materials and Methods**																			
03100	**Concrete and Forms and Accessories**																			
03200	**Concrete Reinforcement**																			
03300	**Cast-In-Place Concrete**																			
03310	**Structural Concrete**																			
	Roller Compacted Concrete																			
EM	11 Mass Placement, 1' Lift, 12" Layer	244,102	CY	1.07	B5	0.009	2,197	91.5	16.91	37,142.61		0.00	6f1	2	102.16	224,433.27		0	261,576	
EM	11 Sloped Face, Nonformed, 1' Lift	198,290	CY	5.00	B5	0.042	8,328	347.0	16.91	140,802.05		0.00	6f1	2	102.16	850,792.80		0	991,595	
EM	11 Roller Compacted Concrete, 1.5"-2" Agg	442,393	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	19,907,672.40				0.00		0	19,907,672	100 lbs of cement by volume
EM	11 Dump Truck (18 CY) Conveying Material	442,393	CY	1.24	C1	0.012	5,309	663.6	11.88	63,067.51		0.00	3e1	1	91.31	484,759.01		0	547,827	(15 Min. Cycles)
EM	11 Truck Mtd. Hydraulic Crane 100 Ton, Co	244,102	CY	0.59	D18a	0.025	6,103	152.6	17.43	106,392.00		0.00	8g	4	6.32	38,543.37		0	144,935	$49.40
EM	11 Surface Prep. Vacuum Truck	*366,153*	SY	0.18	C6a-	0.006	2,197	109.8	17.72	38,925.04		0.00	3j2	1	12.86	28,244.34		0	67,169	
EM	11 Surface Prep. Water Clean	366,153	SY	0.22	C1a	0.008	2,929	122.1	15.63	45,793.60		0.00	3j3	4	11.43	33,485.21		0	79,279	
EM	11 Surface Prep. Water Blast	366,153	SY	0.87	C1a	0.030	10,985	457.7	15.63	171,725.98		0.00	3j4	5	13.25	145,532.83		0	317,259	*$50.45*
EM	**11 Concrete Batch Plant and Delivery**	**455,665**	**CY**	**26.15**	**C6a**	**0.480**	**218,719**	**3,417.5**	**14.44**	**3,157,208.20**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**8,757,034.53**		**0**	**11,914,243**	$77.38
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	164,934	CY	1.07	B5	0.009	1,484	61.9	16.91	25,096.36		0.00	6f1	2	102.16	151,644.10		0	176,740	
EM	11 W/O Vertical Face, Formed, 1' Lift	133,980	CY	7.14	B5	0.060	8,039	335.0	16.91	135,909.31		0.00	6f1	2	102.16	821,228.57		0	957,138	
EM	11 W/O Roller Compacted Concrete, 1.5"-2" Agg	298,914	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	13,451,130.00				0.00		0	13,451,130	100 lbs of cement by volume
EM	11 W/O Dump Truck (18 CY) Conveying Material	298,914	CY	1.24	C1	0.012	3,587	448.4	11.88	42,613.18		0.00	3e1	1	91.31	327,539.87		0	370,153	(15 Min. Cycles)
EM	11 W/O Truck Mtd. Hydraulic Crane 100 Ton, Co	298,914	CY	0.44	D18a	0.025	7,473	186.8	17.43	130,281.67		0.00	8g	0	0.00	0.00		0	130,282	
EM	11 W/O Surface Prep. Vacuum Truck	*247,401*	SY	0.11	C6a-	0.006	1,484	74.2	17.72	26,300.71		0.00	3j2	0	0.00	0.00		0	26,301	
EM	11 W/O Surface Prep. Water Clean	247,401	SY	0.22	C1a	0.008	1,979	82.5	15.63	30,941.62		0.00	3j3	4	11.43	22,625.14		0	53,567	
EM	11 W/O Surface Prep. Water Blast	247,401	SY	0.87	C1a	0.030	7,422	309.3	15.63	116,031.07		0.00	3j4	5	13.25	98,333.00		0	214,364	$51.45
EM	**11 W/O Concrete Batch Plant and Delivery**	**307,881**	**CY**	**26.15**	**C6a**	**0.480**	**147,783**	**2,309.1**	**14.44**	**2,133,248.78**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**5,916,915.22**		**0**	**8,050,164**	$50.47
																				$78.38
	Subtotal Concrete						436,018			$6,401,480		$33,358,802				$17,901,111	$0	$0	$57,661,393	
	Construction Subtotal (Direct Costs)						2,132,054			$33,207,607		$58,589,417				$108,994,160	$13,626,959	$46,165,365	**$260,583,509**	

Indirect Costs				
Sales Tax	6%	of purchased materials + Rental Equipment		10,055,015
Overhead and Profit	16%	of construction cost + general requirements	1.000000	42,233,243
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit		10,950,512
Project Reserve	5%	of construction cost		16,191,114
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation		102,004,018
Construction Subtotal Indirects				**$181,433,902**
Total Construction (Direct and Indirect Costs)				**$442,017,410**
Permits				0
Design	0%	of construction cost		0
Construction Management	0%	of construction cost		0
Total				**$442,017,410**

TABLE 8.13-3

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 3

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No.: 141731.0910
Revision No.: 2
Date: 07/26/05

Opinion Of Probable Cost

EAA Reservoir A-1, Embankment (OG +26) "Inclined Core," With Cutoff Trench, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				60,519	1,033,940	0	6,292,950	0	0	7,326,890	0.8572	6,280,906	13,607,797
	Subtotal				60,519	$1,033,940	$0	$6,292,950	$0	$0	$7,326,890		$6,280,906	$13,607,797
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				187,788	3,085,457	2,297,434	13,173,450	0	0	18,556,341	0.8572	15,907,245	34,463,587
	Subtotal				187,788	$3,085,457	$2,297,434	$13,173,450	$0	$0	$18,556,341		$15,907,245	$34,463,587
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.8572	3,093,014	6,701,121
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,093,014	$6,701,121
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				632,784	9,863,298	5,688,909	42,192,860	0	0	57,745,067	0.8572	49,501,403	107,246,469
	Subtotal				632,784	$9,863,298	$5,688,909	$42,192,860	$0	$0	$57,745,067		$49,501,403	$107,246,469
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				375,240	5,897,990	0	21,765,713	0	52,937,148	80,600,851	0.8572	69,094,304	149,695,156
	Subtotal				375,240	$5,897,990	$0	$21,765,713	$0	$52,937,148	$80,600,851		$69,094,304	$149,695,156
6	**Embankment Construction (Surface Preparation / Cut-off Wall)**													
	Materials and Methods				13,579	239,178	1,844,532	51,563	54,996,480	0	57,131,753	0.8572	48,975,646	106,107,399
	Subtotal				13,579	$239,178	$1,844,532	$51,563	$54,996,480	$0	$57,131,753		$48,975,646	$106,107,399
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				85,152	1,349,448	0	3,379,522	3,188,008	0	7,916,978	0.8572	6,786,753	14,703,731
	Subtotal				85,152	$1,349,448	$0	$3,379,522	$3,188,008	$0	$7,916,978		$6,786,753	$14,703,731
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				265,579	4,069,480	0	16,526,154	0	822,189	21,417,823	0.8572	18,360,222	39,778,045
	Subtotal				265,579	$4,069,480	$0	$16,526,154	$0	$822,189	$21,417,823		$18,360,222	$39,778,045
9	**Embankment Construction (Broken Cap Rock And Silty Sand, Shell, Etc. Soils)**													
	Materials and Methods				110,504	1,691,736	0	3,644,711	0	0	5,336,447	0.8572	4,574,618	9,911,065
	Subtotal				110,504	$1,691,736	$0	$3,644,711	$0	$0	$5,336,447		$4,574,618	$9,911,065
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				29,869	455,659	167,911	1,369,101	0	0	1,992,671	0.8572	1,708,198	3,700,870
	Subtotal				29,869	$455,659	$167,911	$1,369,101	$0	$0	$1,992,671		$1,708,198	$3,700,870
11	**Embankment Construction (Cut-off Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				436,018	6,401,480	33,358,802	17,901,111	0	0	57,661,393	0.8572	49,429,675	107,091,068
	Subtotal				436,018	$6,401,480	$33,358,802	$17,901,111	$0	$0	$57,661,393		$49,429,675	$107,091,068
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.8572	143,620	311,158
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$143,620	$311,158
	Total				**2,231,416**	**$34,581,525**	**$43,357,588**	**$129,475,671**	**$58,184,488**	**$53,862,586**	**$319,461,859**		**$273,855,605**	**$593,317,464**

Case: 3

Opinion Of Probable Cost

January, 2006

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/26/05

EAA Reservoir A-1, Embankment (OG +26) "Inclined Core," With Cutoff Trench, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95	Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75	Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1	**General Requirements**																			
	Mobilization	1	LS	3,194,619						0.00		0.00						3,194,619	3,194,619	
	Supervision	1	LS	15,973,093						0.00		0.00						15,973,093	15,973,093	
	Temporary construction facilities	1	LS	7,986,546						0.00		0.00						7,986,546	7,986,546	
	Temporary utilities	1	LS	4,791,928						0.00		0.00						4,791,928	4,791,928	
	Safety	1	LS	7,986,546						0.00		0.00						7,986,546	7,986,546	
	Miscellaneous	1	LS	6,389,237						0.00		0.00						6,389,237	6,389,237	
	Subtotal Mobilization									$0		$0				$0	$0	$46,321,969	$46,321,969	
2	**Site Work**																			
02225	**Demolition**																			
02230	**Site Clearing**																			
SC	1 Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
SC	1 Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439	
EM	1 Scraper (Strip Peat Embankment A	915555	CY	1.03	B5b	0.006	5,493	274.7	17.17	94,298.47	0.00	0.00	1s1	2	154.87	850,764.54		0	945,063	
EM	1 Surface Prep. (Strip Peat Embankm	1373332	SY	0.35	B5	0.008	10,987	915.6	17.22	189,226.83		0.00	3j4+	2	26	289,566.31		0	478,793	
EM	1 W/O Scraper (Strip Peat Embankment A	618618	CY	1.03	B5b	0.006	3,712	185.6	17.17	63,715.18	0.00	0.00	1s1	2	154.87	574,840.91		0	638,556	
EM	1 W/O Surface Prep. (Strip Peat Embankm	927927	SY	0.35	B5	0.008	7,423	618.6	17.22	127,855.97		0.00	3j4+	2	26	195,652.91		0	323,509	
EM	1 Scraper (Strip Peat Inside Bench A	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
EM	1 W/O Scraper (Strip Peat Inside Bench A	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504	
EM	1 Dozer Angle Blade (Strip Peat Insid	278032	CY	1.15	B2	0.015	4,170	261	16.37	68,271	0.00	0.00	6c	1	120.02	250,278.12		0	318,549	
EM	1 W/O Dozer Angle Blade (Strip Peat Insid	223635	CY	1.15	B2	0.015	3,355	210	16.37	54,914	0.00	0.00	6c	1	120.02	201,311.32		0	256,225	$1.20
02240	**Dewatering**																			
	Seepage Canal																			
SC	2 Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507	
SC	2 12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
	Production Blast																			
SC	4 w/o Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356	
SC	4 w/o 12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
	Cut Off Wall																			
SC	2 w/o Pump,12" Suction (Make-up)	31508	LF	68.94	B4	0.040	1,260	157.5	16.87	21,258.71	0.00	0.00			68.27	2,151,064.86		0	2,172,324	
SC	2 w/o 12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
02300	**Earthwork**																			
02305	**Equipment Mobilization Or Demobilization**																			
	12 Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049	Other: Driving Cost for 140Miles
	12 Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417	Other: Driving Cost for 140Miles
	12 Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12 Crawler Type Drill, 4"	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736	Other: Driving Cost for 140Miles
	12 Air Compressor, 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 Excavator, Diesel Hydraulic, Crawle	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12 Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Concrete Batch Plant, Portable, 2	**8**	**EA**	**3,466**	**C1**	16.000	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12 Concrete Transit Mixer Truck	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12 Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12 Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12 Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12 Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Wheeled Skid Steer, Diesel, w/ Bro	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12 Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12 Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310	**Grading**																			
02315	**Excavation and Fill**			13																
SC	2 Drilling and Blasting (Seepage Can	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2 Excavating Cap Rock (Seepage Ca	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2 Dump Truck (Canal/Stock Pile Area	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2 Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2 Dozer Angle Blade (Stock Pile Area	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
				13.32																***18,556,341***
SC	3 Excavated - Silty, Sand, Shells (See	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3 Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812	
SC	3 Haul to Dewater and Work Stock Pi	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719	(Avg. 1,000 FT. Around Trip)
SC	3 Dozer Angle Blade - Work Stock Pil	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790	***3,608,106***
EM	4 Drilling and Blasting (Prod. Blast Ar	1,668,193	CY	6.02	B6	0.080	133,455	5,560.6	17.11	2,283,867.22	1.89	3,152,884.47	6h	4	34.56	4,612,817.38		0	10,049,569	
EM	4 Excavating Cap Rock (Prod. Blast A	1,668,193	CY	1.75	B2	0.015	24,668	1,541.8	16.37	403,821.74	0.00	0.00	1b	1	102.03	2,516,993.84		0	2,920,816	
EM	4 Dump Truck (Prod. Blast Stock Pile	2,085,241	CY	1.71	C1	0.017	34,555	4,319.4	11.88	410,518.43	0.00	0.00	3e1	1	91.31	3,155,388.84		0	3,565,907	(Avg. 2 Miles Around Trip)
EM	4 Hoe Ram (Stock Pile Area)	218,950	CY	1.74	A10	0.077	16,859	1,685.9	17.89	301,610.62	0.00	0.00	6i	1	5.91	79,737.15		0	381,348	
EM	4 Dozer Angle Blade (Prod. Blast Are	2,085,241	CY	1.20	B2	0.016	32,843	2,052.7	16.37	537,632.49	0.00	0.00	6c	1	120.02	1,970,940.20		0	2,508,573	19,426,212
EM	4 Excavated - Silty, Sand, Shells (Pro	3,075,717	CY	0.80	B3	0.005	15,379	640.8	16.75	257,591.29	0.00	0.00	1n	1	144.10	2,216,102.44		0	2,473,694	
EM	4 Haul to Dewater and Work Stock Pi	3,383,289	CY	1.71	C1	0.017	56,066	7,008.2	11.88	666,063.20	0.00	0.00	3e1	1	91.31	5,119,595.72		0	5,785,659	(Avg. 2 Miles Around Trip)
EM	4 Dozer Angle Blade (Work Stock Pile	3,383,289	CY	0.69	B2	0.009	30,450	1,903.1	16.37	498,459.91	0.00	0.00	6c	1	120.02	1,827,335.04		0	2,325,795	10,585,148
																				32,291,293
EM	4 W/O Drilling and Blasting (Prod. Blast Ar	1,341,812	CY	6.02	B6	0.080	107,345	4,472.7	17.11	1,837,029.66	1.89	2,536,024.09	6h	4	34.56	3,710,321.80		0	8,083,376	
EM	4 W/O Excavating Cap Rock (Prod. Blast A	1,341,812	CY	1.75	B2	0.015	19,842	1,240.1	16.37	324,814.20	0.00	0.00	1b	1	102.03	2,024,545.16		0	2,349,359	
EM	4 W/O Dozer Angle Blade (Prod. Blast Are	1,610,174	CY	1.20	B2	0.016	25,360	1,585.0	16.37	415,147.14	0.00	0.00	6c	1	120.02	1,521,913.60		0	1,937,061	12,369,796
																				45,374,016

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/26/05

Case: 3
Opinion Of Probable Cost

EAA Reservoir A-1, Embankment (OG +26) "Inclined Core," With Cutoff Trench, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

					Labor						Material		Equipment							
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	4 W/O Excavated - Silty, Sand, Shells (Pro	3,801,800	CY	0.80	B3	0.005	19,009	792.0	16.75	318,400.73	0.00	0.00	1n	1	144.10	2,739,256.54		0	3,057,657	
EM	4 W/O Haul to Dewater and Work Stock Pi	4,181,980	CY	1.71	C1	0.017	69,301	8,662.7	11.88	823,300.38	0.00	0.00	3e1	1	91.31	6,328,175.92		0	7,151,476	(Avg. 2 Miles Around Trip)
EM	4 W/O Dozer Angle Blade (Work Stock Pile	4,181,980	CY	0.69	B2	0.009	37,638	2,352.4	16.37	616,131.08	0.00	0.00	6c	1	120.02	2,258,713.03		0	2,874,844	13,083,978
EM	5 Crusher (Transition Borrow Area)	1,440,220	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		26,139,999	26,139,999	
EM	5 Dozer Loader (Transition Borrow A	1,440,220	CY	0.82	B5	0.013	18,363	1,530.2	17.22	316,268.78	0.00	0.00	6j	1	47.46	871,436.45		0	1,187,705	
EM	5 Dump Truck (Transition Borrow Are	2,016,308	CY	1.71	C1	0.017	33,413	4,176.6	11.88	396,947.76	0.00	0.00	3e1	1	91.31	3,051,079.95		0	3,448,028	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankm	2,016,308	CY	1.03	B2	0.014	27,220	1,701.3	16.37	445,594.09	0.00	0.00	6c	1	120.02	1,633,530.93		0	2,079,125	32,854,857
EM	5 Cruncher (Transition Borrow Area)	50,413	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		915,004	915,004	
EM	5 Dozer Loader (Transition Borrow A	50,413	CY	0.82	B5	0.013	643	53.6	17.22	11,070.67	0.00	0.00	6j	1	47.46	30,503.74		0	41,574	
EM	5 Dump Truck (Transition Borrow Are	70,579	CY	1.71	C1	0.017	1,170	146.2	11.88	13,894.75	0.00	0.00	3e1	1	91.31	106,799.93		0	120,695	(Avg. 2 Miles Around Trip)
EM	5 Backhoe (Vertical New Embankme	70,579	CY	3.28	B4a	0.153	10,799	225.0	17.08	184,457.38	0.00	0.00	2	1	4.33	46,748.76		0	231,206	1,308,479
EM	5 Cruncher (Transition Borrow Area)	1,329,082	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		24,122,831	24,122,831	
EM	5 Dozer Loader (Transition Borrow A	1,329,082	CY	0.82	B5	0.013	16,946	1,412.1	17.22	291,863.00	0.00	0.00	6j	1	47.46	804,189.56		0	1,096,053	
EM	5 Dump Truck (Transition Borrow Are	1,860,714	CY	1.71	C1	0.017	30,835	3,854.3	11.88	366,316.15	0.00	0.00	3e1	1	91.31	2,815,634.64		0	3,181,951	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (Sloped New Er	1,860,714	CY	8.18	B2	0.107	199,282	12,455.2	16.37	3,262,254.44	0.00	0.00	6c	1	120.02	11,959,300.41		0	15,221,555	43,622,389
EM	5 Crusher (Granular Toe Borrow Area	95,098	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,759,314	1,759,314	
EM	5 Dozer Loader (Granular Toe Borrov	95,098	CY	0.82	B5	0.013	1,213	101.0	17.22	20,883.30	0.00	0.00	6j	1	47.46	57,541.15		0	78,424	
EM	5 Dump Truck (Granular Toe Borrow	133,137	CY	1.71	C1	0.017	2,206	275.8	11.88	26,210.55	0.00	0.00	3e1	1	91.31	201,463.51		0	227,674	(Avg. 2 Miles Around Trip)
EM	5 Backhoe Loader (New Embankmen	133,137	CY	3.84	B4-	0.160	21,302	887.6	16.65	354,748.81	0.00	0.00	2a	1	7.33	156,174.10		0	510,923	
EM	5 Compacting (New Embankment Are	133,137	CY	1.79	A1	0.089	11,849	1,481.2	17.51	207,479.86	0.00	0.00	8b2	1	2.64	31,310.33		0	238,790	2,815,126
																				80,600,851
SC	6 Keyway (Cut Through Cap Rock)	114,576	LF	480.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	54,996,480	0	54,996,480	(Assumed for 2' Wide at $40/LF)
SC	6 w/o Shotcrete Keyway (Cap Rock)	22,632	CY	94.35	D8a	0.600	13,579	282.9	17.61	239,178.11	81.50	1,844,532.15	8c	1	3.80	51,562.67		0	2,135,273	
EM	9 Dozer and Dump Trucks (Select Fil	2,662,746	CY	0.74	C6	0.024	62,575	2,234.8	14.17	886,681.18	0.00	0.00	3f1	2	60.38	1,079,538.18		0	1,966,219	
EM	9 Dozer Angle Blade (Select Fill Belo	2,662,746	CY	0.69	B2	0.009	23,965	1,497.8	16.37	392,302.40	0.00	0.00	6c	1	120.02	1,438,165.65		0	1,830,468	
EM	9 Compact (Select Fill Below Drain)	2,662,746	CY	0.58	B5	0.009	23,965	1,997.1	17.22	412,752.29	0.00	0.00	6g1	1	70.54	1,127,007.19		0	1,539,759	5,336,447
EM	7 Dozer Loader (Sand Horiz. Filter)	382,684	CY	0.46	B2	0.006	2,296	143.5	16.37	37,587.21	0.00	0.00	6c	1	120.02	137,793.27		0	175,380	
EM	7 Dump Truck (Sand Horiz. Filter)	382,684	CY	0.93	C1	0.009	3,444	430.5	11.88	40,916.56	0.00	0.00	3e1	1	91.31	314,499.02		0	355,416	
EM	7 Wash and Screen (Sand Horiz. Filte	382,684	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	1,503,947	0	1,503,947	
EM	7 Dozer Angle Blade (Sand Horiz. Fil	382,684	CY	0.69	B2	0.009	3,444	215.3	16.37	56,380.81	0.00	0.00	6c	1	120.02	206,689.90		0	263,071	
EM	7 Dozer and Dump Trucks (Sand Hor	382,684	CY	1.75	C6	0.024	8,993	321.2	14.17	127,431.81	0.00	0.00	3f1	2	60.38	543,020.71		0	670,453	
EM	7 Dozer Angle Blade (Sand Horiz. Fil	382,684	CY	0.69	B2	0.009	3,444	215.3	16.37	56,380.81	0.00	0.00	6c	1	120.02	206,689.90		0	263,071	
EM	7 Compact (Sand Horiz. Filter)	382,684	CY	0.79	B5	0.009	3,444	287.0	17.22	59,319.82	0.00	0.00	6g1	1	70.54	242,956.37		0	302,276	
EM	7 Dozer Loader (Sand Horiz. Drain)	191,342	CY	0.46	B2	0.006	1,148	71.8	16.37	18,793.60	0.00	0.00	6c	1	120.02	68,896.63		0	87,690	
EM	7 Dump Truck (Sand Horiz. Drain)	191,342	CY	0.93	C1	0.009	1,722	215.3	11.88	20,458.28	0.00	0.00	3e1	1	91.31	157,249.51		0	177,708	
EM	7 Wash and Screen (Sand Horiz. Dra	191,342	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	751,974	0	751,974	
EM	7 Dozer Angle Blade (Sand Horiz. Dr	191,342	CY	0.69	B2	0.009	1,722	107.6	16.37	28,190.41	0.00	0.00	6c	1	120.02	103,344.95		0	131,535	
EM	7 Dozer and Dump Trucks (Sand Hor	191,342	CY	1.75	C6	0.024	4,497	160.6	14.17	63,715.90	0.00	0.00	3f1	2	60.38	271,510.36		0	335,226	
EM	7 Dozer Angle Blade (Sand Horiz. Dr	191,342	CY	0.69	B2	0.009	1,722	107.6	16.37	28,190.41	0.00	0.00	6c	1	120.02	103,344.95		0	131,535	
EM	7 Compact (Sand Horiz. Drain)	191,342	CY	0.79	B5	0.009	1,722	143.5	17.22	29,659.91	0.00	0.00	6g1	1	70.54	121,478.18		0	151,138	
EM	7 Dozer Loader (Sand Vert. Filter)	237,172	CY	0.46	B2	0.006	1,423	88.9	16.37	23,295.07	0.00	0.00	6c	1	120.02	85,398.82		0	108,694	
EM	7 Dump Truck (Sand Vert. Filter)	237,172	CY	0.93	C1	0.009	2,135	266.8	11.88	25,358.46	0.00	0.00	3e1	1	91.31	194,914.07		0	220,273	
EM	7 Wash and Screen (Sand Vert. Filter	237,172	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	932,087	0	932,087	
EM	7 Dozer Angle Blade (Sand Vert. Filte	237,172	CY	0.69	B2	0.009	2,135	133.4	16.37	34,942.60	0.00	0.00	6c	1	120.02	128,098.23		0	163,041	
EM	7 Dozer and Dump Trucks (Sand Ver	237,172	CY	1.75	C6	0.024	5,574	199.1	14.17	78,977.20	0.00	0.00	3f1	2	60.38	336,542.78		0	415,520	
EM	7 Backhoe (Sand Vert. Filter)	237,172	CY	3.28	B4a	0.153	36,287	756.0	17.08	619,848.67	0.00	0.00	2	1	4.33	157,094.05		0	776,943	
EM	8 Dozer and Dump Trucks (Max. 6" R	0	CY	0.00	C6	0.024	0	0.0	14.17	0.00	0.00	0.00	3f1	2	60.38	0.00		0	0	
EM	8 Dozer With Ripper Attach, (Max. 6"	0	CY	0.00	B5	0.020	0	0.0	17.22	0.00	0.00	0.00	6g	2	84.81	0.00		0	0	
EM	8 Dozer Angle Blade (Max. 6" Rando	0	CY	0.00	B2	0.009	0	0.0	16.37	0.00	0.00	0.00	6c	1	120.02	0.00		0	0	
EM	8 Compact (Max. 6" Random Fill)	0	CY	0.00	B5	0.009	0	0.0	17.22	0.00	0.00	0.00	6g1	1	70.54	0.00		0	0	
EM	8 W/O Dozer and Dump Trucks (Max. 6" R	0	CY	0.00	C6	0.024	0	0.0	14.17	0.00	0.00	0.00	3f1	2	60.38	0.00		0	0	
EM	8 W/O Dozer With Ripper Attach, (Max. 6"	0	CY	0.00	B5	0.020	0	0.0	17.22	0.00	0.00	0.00	6g	2	84.81	0.00		0	0	
EM	8 W/O Dozer Angle Blade (Max. 6" Rando	0	CY	0.00	B2	0.009	0	0.0	16.37	0.00	0.00	0.00	6c	1	120.02	0.00		0	0	
EM	8 W/O Compact (Max. 6" Random Fill)	0	CY	0.00	B5	0.009	0	0.0	17.22	0.00	0.00	0.00	6g1	1	70.54	0.00		0	0	
EM	8 Dozer and Dump Trucks (Mass Rar	3,764,783	CY	1.75	C6	0.024	88,472	3,159.7	14.17	1,253,653.77	0.00	0.00	3f1	2	60.38	5,342,151.11		0	6,595,805	
EM	8 Dozer Angle Blade (Mass Random	3,764,783	CY	0.69	B2	0.009	33,883	2,117.7	16.37	554,665.41	0.00	0.00	6c	1	120.02	2,033,382.26		0	2,588,048	
EM	8 Compact (Mass Random Fill)	3,764,783	CY	0.79	B5	0.009	33,883	2,823.6	17.22	583,578.94	0.00	0.00	6g1	1	70.54	2,390,166.00		0	2,973,745	12,157,598
EM	8 W/O Dozer and Dump Trucks (Mass Rar	2,543,772	CY	1.75	C6	0.024	59,779	2,135.0	14.17	847,063.36	0.00	0.00	3f1	2	60.38	3,609,561.56		0	4,456,625	
EM	8 W/O Dozer Angle Blade (Mass Random	2,543,772	CY	0.69	B2	0.009	22,894	1,430.9	16.37	374,773.93	0.00	0.00	6c	1	120.02	1,373,906.93		0	1,748,681	
EM	8 W/O Compact (Mass Random Fill)	2,543,772	CY	0.79	B5	0.009	22,894	1,907.8	17.22	394,310.10	0.00	0.00	6g1	1	70.54	1,614,977.03		0	2,009,287	20,372,190
EM	10 Common Borrow (Top Soil - Peat)	222,906	CY	3.65	C6	0.047	10,477	374.2	14.17	148,453.01	0.00	0.00	3f	3	63.57	666,010.57		0	814,464	
EM	10 W/O Common Borrow (Top Soil - Peat)	150,612	CY	3.65	C6	0.047	7,079	252.8	14.17	100,306.09	0.00	0.00	3f	3	63.57	450,007.14		0	550,313	1,364,777
02370	**Erosion and Sedimentation Control**																			
02600	**Drainage and Containment**																			
02620	**Subdrainage System (Seepage Water)**																			
02700	**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o Crusher (Transition Borrow Area)	65775	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		822,189	822,189	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" T	197325	SY	1.13	B7a	0.019	3,774	52.4	16.28	61,434.04	0.00	0.00	7a+	7	42.93	162,009.17		0	223,443	1,045,632

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/26/05

Case: 3
Opinion Of Probable Cost

EAA Reservoir A-1, Embankment (OG +26) "Inclined Core," With Cutoff Trench, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

					Labor						Material		Equipment							
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
02910	**Plant Preparation**																			
EM	10 Fine Grading	668,034	SY	0.32	B5a	0.008	5,344	334.0	17.30	92,429.12	0.00	0.00	7d	1	22.97	122,759.39		0	215,189	
EM	10 W/O Fine Grading	451,374	SY	0.32	B5a	0.008	3,611	225.7	17.30	62,452.11	0.00	0.00	7d	1	22.97	82,945.53		0	145,398	
02920	**Lawns and Grasses**																			
EM	10 Hydro or Air Seeding w/ Mulch and	668,034	SY	0.24	C4a	0.003	2,004	3.0	15.49	31,043.52	0.15	100,205.03	3j5	2	14.11	28,274.17		0	159,523	520,109
EM	10 W/O Hydro or Air Seeding w/ Mulch and	451,374	SY	0.24	C4a	0.003	1,354	3.0	15.49	20,975.35	0.15	67,706.10	3j5	2	14.11	19,104.17		0	107,786	627,894
	Subtotal Site Construction						1,795,398			$28,180,045		$9,998,785				$111,574,560	$58,184,488	$53,862,586	$261,800,465	
3	**Concrete**																			
03050	**Basic Concrete Materials and Methods**																			
03100	**Concrete and Forms and Accessories**																			
03200	**Concrete Reinforcement**																			
03300	**Cast-In-Place Concrete**																			
03310	**Structural Concrete**																			
03370	**Specially Placed Concrete**																			
	Roller Compacted Concrete																			
EM	11 Mass Placement, 1' Lift, 12" Layer	244,102	CY	1.07	B5	0.009	2,197	91.5	16.91	37,142.61		0.00	6f1	2	102.16	224,433.27		0	261,576	
EM	11 Sloped Face, Nonformed, 1' Lift	198,290	CY	5.00	B5	0.042	8,328	347.0	16.91	140,802.05		0.00	6f1	2	102.16	850,792.80		0	991,595	
EM	11 Roller Compacted Concrete, 1.5"-2'	442,393	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	19,907,672.40				0.00		0	19,907,672	100 lbs of cement by volume
EM	11 Dump Truck (18 CY) Conveying Ma	442,393	CY	1.24	C1	0.012	5,309	663.6	11.88	63,067.51		0.00	3e1	1	91.31	484,759.01		0	547,827	(15 Min. Cycles)
EM	11 Truck Mtd. Hydraulic Crane 100 To	244,102	CY	0.59	D18a	0.025	6,103	152.6	17.43	106,392.00		0.00	8g	4	6.32	38,543.37		0	144,935	$49.40
EM	11 Surface Prep. Vacuum Truck	*366,153*	SY	0.18	C6a-	0.006	2,197	109.8	17.72	38,925.04		0.00	3j2	1	12.86	28,244.34		0	67,169	
EM	11 Surface Prep. Water Clean	366,153	SY	0.22	C1a	0.008	2,929	122.1	15.63	45,793.60		0.00	3j3	4	11.43	33,485.21		0	79,279	
EM	11 Surface Prep. Water Blast	366,153	SY	0.87	C1a	0.030	10,985	457.7	15.63	171,725.98		0.00	3j4	5	13.25	145,532.83		0	317,259	*$50.45*
EM	**11 Concrete Batch Plant and Deliver**	**455,665**	**CY**	**26.15**	**C6a**	**0.480**	**218,719**	**3,417.5**	**14.44**	**3,157,208.20**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**8,757,034.53**		**0**	**11,914,243**	$77.38
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	164,934	CY	1.07	B5	0.009	1,484	61.9	16.91	25,096.36		0.00	6f1	2	102.16	151,644.10		0	176,740	
EM	11 W/O Vertical Face, Formed, 1' Lift	133,980	CY	7.14	B5	0.060	8,039	335.0	16.91	135,909.31		0.00	6f1	2	102.16	821,228.57		0	957,138	
EM	11 W/O Roller Compacted Concrete, 1.5"-2'	298,914	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	13,451,130.00				0.00		0	13,451,130	100 lbs of cement by volume
EM	11 W/O Dump Truck (18 CY) Conveying Ma	298,914	CY	1.24	C1	0.012	3,587	448.4	11.88	42,613.18		0.00	3e1	1	91.31	327,539.87		0	370,153	(15 Min. Cycles)
EM	11 W/O Truck Mtd. Hydraulic Crane 100 To	298,914	CY	0.44	D18a	0.025	7,473	186.8	17.43	130,281.67		0.00	8g	0	0.00	0.00		0	130,282	
EM	11 W/O Surface Prep. Vacuum Truck	*247,401*	SY	0.11	C6a-	0.006	1,484	74.2	17.72	26,300.71		0.00	3j2	0	0.00	0.00		0	26,301	
EM	11 W/O Surface Prep. Water Clean	247,401	SY	0.22	C1a	0.008	1,979	82.5	15.63	30,941.62		0.00	3j3	4	11.43	22,625.14		0	53,567	
EM	11 W/O Surface Prep. Water Blast	247,401	SY	0.87	C1a	0.030	7,422	309.3	15.63	116,031.07		0.00	3j4	5	13.25	98,333.00		0	214,364	$51.45
EM	**11 W/O Concrete Batch Plant and Deliver**	**307,881**	**CY**	**26.15**	**C6a**	**0.480**	**147,783**	**2,309.1**	**14.44**	**2,133,248.78**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**5,916,915.22**		**0**	**8,050,164**	$50.47
																				$78.38
	Subtotal Concrete						436,018			$6,401,480		$33,358,802				$17,901,111	$0	$0	$57,661,393	
	Construction Subtotal (Direct Costs)						2,231,416			$34,581,525		$43,357,588				$129,475,671	$58,184,488	$100,184,556	**$365,783,828**	

Indirect Costs

Sales Tax	6%	of purchased materials + Rental Equipment	10,369,996
Overhead and Profit	12%	of construction cost + general requirements	43,812,175
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	14,698,810
Project Reserve	5%	of construction cost	21,733,240
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	136,919,415
Construction Subtotal Indirects			**$227,533,636**
Total Construction (directs and Indirects)			**$593,317,464**
Permits			0
Design	0%	of construction cost	0
Construction Management	0%	of construction cost	0
Total			**$593,317,464**

TABLE 8.13-4

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 4

Case: 4

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02

Project No.: 141731.0910

Revision No.: 1

Date: 07/25/05

Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG +25) USACE Design "Including 2.5 Feet Parapet Wall," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials & Methods				55,793	957,276	0	6,943,431	0	0	7,900,707	1.0221	8,075,296	15,976,003
	Subtotal				55,793	$957,276	$0	$6,943,431	$0	$0	$7,900,707		$8,075,296	$15,976,003
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials & Methods				186,264	3,059,757	2,297,434	11,810,334	0	0	17,167,525	1.0221	17,546,891	34,714,416
	Subtotal				186,264	$3,059,757	$2,297,434	$11,810,334	$0	$0	$17,167,525		$17,546,891	$34,714,416
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				37,359	541,910	0	3,562,699	0	0	4,104,608	1.0221	4,195,311	8,299,920
	Subtotal				37,359	$541,910	$0	$3,562,699	$0	$0	$4,104,608		$4,195,311	$8,299,920
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				95,230	1,553,329	1,106,693	5,736,228	0	0	8,396,250	1.0221	8,581,790	16,978,040
	Subtotal				95,230	$1,553,329	$1,106,693	$5,736,228	$0	$0	$8,396,250		$8,581,790	$16,978,040
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				0	0	0	0	0	0	0	1.0221	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				204,014	3,202,598	12,262,624	4,590,538	97,562,784	0	117,618,544	1.0221	120,217,667	237,836,212
	Subtotal				204,014	$3,202,598	$12,262,624	$4,590,538	$97,562,784	$0	$117,618,544		$120,217,667	$237,836,212
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				0	0	0	0	0	0	0	1.0221	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				3,291	55,387	0	121,384	0	806,667	983,438	1.0221	1,005,169	1,988,607
	Subtotal				3,291	$55,387	$0	$121,384	$0	$806,667	$983,438		$1,005,169	$1,988,607
9	**Embankment Construction (Broken Cap Rock And Silty Sand, Shell, Etc. Soils)**													
	Materials and Methods				0	0	0	0	0	0	0	1.0221	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				0	0	0	0	0	0	0	1.0221	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
11	**Embankment Construction (RCC and Concrete)**													
	Materials and Methods				3,072,337	48,167,967	89,366,825	65,758,595	0	0	203,293,387	1.0221	207,785,745	411,079,132
	Subtotal				3,072,337	$48,167,967	$89,366,825	$65,758,595	$0	$0	$203,293,387		$207,785,745	$411,079,132
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				737	9,103	0	43,887	0	87,069	140,059	1.0221	143,154	283,213
	Subtotal				737	$9,103	$0	$43,887	$0	$87,069	$140,059		$143,154	$283,213
	Total				**3,655,025**	**$57,547,327**	**$105,033,576**	**$98,567,095**	**$97,562,784**	**$893,736**	**$359,604,518**		**$367,551,024**	**$727,155,541**

EAA Reservoir A-1 Basis of Design Report | **Case: 4** | January, 2006

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 1
Date: 07/25/05

EAA Reservoir A-1, RCC (OG +25) USACE Design "Including 2.5 Feet Parapet Wall," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
	1	**General Requirements**																			
		Mobilization	1	LS	3,596,045						0.00		0.00						3,596,045	3,596,045	
		Supervision	1	LS	17,980,226						0.00		0.00						17,980,226	17,980,226	
		Temporary Construction Facilities	1	LS	8,990,113						0.00		0.00						8,990,113	8,990,113	
		Temporary Utilities	1	LS	5,394,068						0.00		0.00						5,394,068	5,394,068	
		Safety	1	LS	8,990,113						0.00		0.00						8,990,113	8,990,113	
		Miscellaneous	1	LS	7,192,090						0.00		0.00						7,192,090	7,192,090	
		Subtotal Mobilization									$0		$0				$0	$0	$52,142,655	$52,142,655	
	2	**Site Work**																			
02225		**Demolition**																			
02230		**Site Clearing**																			
SC	1	Scraper (Strip Peat Canal Area)	860,444	CY	2.06	B5b	0.012	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
SC	1	Scraper (Strip Peat Bench Area)	1720889	CY	1.55	B5b	0.009	15,488	774.4	17.17	265,867.01	0.00	0.00	1s1	2	154.87	2,398,662.81		0	2,664,530	
EM	1	Scraper (Strip Peat Embankment Area)	1010592	CY	1.03	B5b	0.006	6,064	303.2	17.17	104,086.93	0.00	0.00	1s1	2	154.87	939,076.49		0	1,043,163	
EM	1	Surface Prep. (Strip Peat Embankment	1515888	SY	0.35	B5	0.008	12,127	1,010.6	17.22	208,869.15		0.00	3j4+	2	26	319,624.17		0	528,493	
EM	1	Scraper (Strip Peat Inside Bench Area)	2581333	CY	0.69	B5b	0.004	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
EM	1	Dozer Angle Blade (Strip Peat Inside Be	97592	CY	1.15	B2	0.015	1,464	91	16.37	23,964	0.00	0.00	6c	1	120.02	87,850		0	111,814	
02240		**Dewatering**																			
		Seepage Canal																			
SC	2	Pump,12" Suction (Make-up)	38,914	LF	40.86	B4	0.040	1,557	194.6	16.87	26,255.00	0.00	0.00			40.19	1,563,780.00		0	1,590,035	
SC	2	12" Dia Pipe 2,500 GPM	54,595	LF	5.48	B4	0.098	5,350	668.8	16.87	90,246.68	0.00	0.00			3.83	209,099.62		0	299,346	
		Production Blast																			
SC	4	Pump,12" Suction (Make-up)	26136	LF	40.86	B4	0.040	1,045	130.7	16.87	17,633.95	0.00	0.00			40.19	1,050,300.00		0	1,067,934	
SC	4	12" Dia Pipe 2,500 GPM	92928	LF	5.48	B4	0.098	9,107	1,138.4	16.87	153,611.38	0.00	0.00			3.83	355,914.24		0	509,526	
		Cutoff Wall																			
SC	2 w/o	Pump,12" Suction (Make-up)	*31944*	LF	40.86	B4	0.040	1,278	159.7	16.87	21,552.61	0.00	0.00			40.19	1,283,700.00		0	1,305,253	
SC	2 w/o	12" Dia Pipe 2,500 GPM	*72,019*	LF	5.48	B4	0.098	7,058	882.2	16.87	119,048.82	0.00	0.00			3.83	275,833.54		0	394,882	
02300		**Earthwork**																			
02305		**Equipment Mobilization Or Demobilization**																			
	12	Dump Truck (26 Tons)	12	EA	346	C1	2.000	24	3.0	11.88	285.12	0.00	0.00	3e1	1	49.87	1,196.96		2,668	4,150	Other: Driving Cost for 140Miles
	12	Dozers (Above 150 HP)	28	EA	578	C1	2.667	75	9.3	11.88	887.15	0.00	0.00	3g	2	65.48	4,889.41		10,398	16,174	Other: Driving Cost for 140Miles
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12	Crawler Type Drill, 4"	16	EA	322	C2	6.000	96	12.0	15.55	1,492.80	0.00	0.00	6h	4	34.56	3,318.19		346	5,157	Other: Driving Cost for 140Miles
	12	Air Compressor, 600 CFM	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	50 Ft Air Hose, 3" Dia.	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	Excavator, Diesel Hydraulic, Crawler M	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	**Concrete Batch Plant, Portable, 200 (**	**8**	**EA**	**3,466**	**C1**	16.000	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12	Scraper, Self-Propelled, 32-44 CY	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane up To 75 Ton	12	EA	481	C1	2.222	27	3.3	11.88	316.77	0.00	0.00	3g	2	65.48	1,745.83		3,713	5,775	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12	Attachment Clam Bucket, 1/2 CY	12	EA	578	C1	2.667	32	4.0	11.88	380.21	0.00	0.00	3g	2	65.48	2,095.46		4,456	6,932	Other: Driving Cost for 140Miles
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Rubber tired backhoe-loader, 3/4 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Wheeled Skid Steer, Diesel, w/ Broom	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Wash and Screen (Sand Horiz. Blanket	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310		**Grading**																			
02315		**Excavation and Fill**																			
SC	2	Drilling & Blasting (Seepage Canal Are	**1,215,573**	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2	Excavating Cap Rock (Seepage Canal	**1,215,573**	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2	Dump Truck (Canal/Stock Pile Areas)	**1,519,467**	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2	Hoe Ram (Stock Pile Area)	**159,544**	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2	Dozer Angle Blade (Stock Pile Area)	**1,519,467**	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
																					17,167,525
SC	3	Excavated - Silty, Sand, Shells (Seepag	**958,372**	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3	Cut Through Limestone	**193,406**	CY	4.02	B3	0.025	4,835	201.5	16.75	80,988.93	0.00	0.00	1n	1	144.10	696,761.77		0	777,751	
SC	3	Haul to Dewater & Work Stock Piles (S	**1,266,956**	CY	1.33	C1	0.013	16,330	2,041.2	11.88	193,996.33	0.00	0.00	3e1	1	91.31	1,491,123.94		0	1,685,120	(Avg. 1,000 FT. Around Trip)
SC	3	Hauling (5 Miles)	0	CY	0.00	C1	0.040	0	0.0	11.88	0.00	0.00	0.00	3e	1	49.87	0.00		0	0	
SC	3	Dozer Angle Blade - Work Stock Piles (	**1,266,956**	CY	0.69	B2	0.009	11,403	712.7	16.37	186,660.65	0.00	0.00	6c	1	120.02	684,290.84		0	870,951	***4,104,608***
EM	4	Drilling and Blasting (Prod. Blast Area)	**585,552**	CY	6.02	B6	0.080	46,844	1,951.8	17.11	801,659.72	1.89	1,106,693.28	6h	4	34.56	1,619,144.00		0	3,527,497	
EM	4	Excavating Cap Rock (Prod. Blast Area	**585,552**	CY	1.75	B2	0.015	8,659	541.2	16.37	141,745.38	0.00	0.00	1b	1	102.03	883,489.45		0	1,025,235	
EM	4	Dump Truck (Prod. Blast Stock Pile Are	**731,940**	CY	1.71	C1	0.017	12,129	1,516.2	11.88	144,095.98	0.00	0.00	3e1	1	91.31	1,107,572.34		0	1,251,668	(Avg. 2 Miles Around Trip)
EM	4	Hoe Ram (Stock Pile Area)	**76,854**	CY	1.74	A10	0.077	5,918	591.8	17.89	105,868.28	0.00	0.00	6i	1	5.91	27,988.52		0	133,857	
EM	4	Dozer Angle Blade (Prod. Blast Area)	**731,940**	CY	1.20	B2	0.016	11,528	720.5	16.37	188,714.26	0.00	0.00	6c	1	120.02	691,819.28		0	880,534	6,818,790
EM	6	Clean Cap Rock Surface (Embankment	297,370	SY	0.87	C1a	0.030	8,921	371.7	15.63	139,466.34		0.00	3j4	5	13.25	118,193.72		0	**257,660**	
EM	6	Cement Grout Cap Rock (Embankment	48,787	CY	78.98	B4	1.500	73,181	2,286.9	16.87	1,234,377.14	45.00	2,195,424.00	3i	2	5.78	423,183.35		0	**3,852,984**	
EM	**6**	**Concrete Batch Plant and Delivery**	**50,251**	**CY**	**26.15**	**C6a**	**0.480**	**24,120**	**376.9**	**14.44**	**348,177.85**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**965,728.36**		**0**	**1,313,906**	

EAA Reservoir A-1 Basis of Design Report — Case: 4 — January, 2006

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 1
Date: 07/25/05

EAA Reservoir A-1, RCC (OG +25) USACE Design "Including 2.5 Feet Parapet Wall," with 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	6 w/o	Cutoff Wall (Embankment Area)	116,160	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	9,292,800	0	9,292,800	(Quoted for 2' Wide at $40/LF)
EM	6 w/o	Lean Concrete Fill In Cap Rock Cutoff \	154,880	CY	67.54	D8	0.137	21,219	442.1	17.69	375,250.23	65.00	10,067,200.00	8b1	2	0.83	17,628.35		0	10,460,079	
EM	6 w/o	**Concrete Batch Plant and Delivery**	**159,526**	**CY**	**26.15**	**C6a**	**0.480**	**76,573**	**1,196.4**	**14.44**	**1,105,326.52**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,065,804.31**		**0**	**4,171,131**	
EM	6 w/o	Cutoff Wall (Embankment Area)	3,484,800	SF	25.00	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	87,120,000	0	87,120,000	(Plastic Concrete)
SC	6 w/o	Cutoff Wall (Through Limestone)	25,555	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	1,149,984	0	1,149,984	***117,618,544***
02700		**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o	Crusher (Transition Borrow Area)	***64533***	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		806,667	806,667	
EM	8 w/o	Aggregate Base 1-1/2" Stone, 12" Thicl	*193600*	SY	0.91	B7a	0.017	3,291	51.4	16.83	55,386.78	0.00	0.00	7a+	6	36.88	121,384.06		0	176,771	983,438
		Subtotal Site Construction						582,689			$9,379,360		$15,666,751				$32,808,500	$97,562,784	$893,736	$156,311,130	
3		**Concrete**																			
03050		**Basic Concrete Materials and Methods**																			
03100		**Concrete and Forms and Accessories**																			
03200		**Concrete Reinforcement**																			
03300		**Cast-In-Place Concrete**																			
03310		**Structural Concrete**																			
EM	11	Parapet Wall Footing	0	CY	0.00	D8b	1.382	0	0.0	17.59	0.00	199.85	0.00	8b	3	8.93	0.00		0	0	
EM	11	Parapet Wall	52,272	CY	174.72	D8b	2.182	114,058	1,782.1	17.59	2,005,986.35	116.85	6,107,983.20	8b	3	8.93	1,018,873.26		0	9,132,843	
EM	**11**	**Concrete Batch Plant and Delivery**	**53,840**	**CY**	**26.15**	**C6a**	**0.480**	**25,843**	**403.8**	**14.44**	**373,047.70**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,034,708.95**		**0**	**1,407,757**	$201.65
03370		**Specially Placed Concrete**																			
		Roller Compacted Concrete																			
EM	11	Mass Placement, 1' Lift, 12" Layer	1,452,000	CY	1.07	B5	0.009	13,068	544.5	16.91	220,936.32		0.00	6f1	2	102.16	1,335,002.11		0	1,555,938	
EM	11	Vertical Face, Formed, 1' Lift	232,320	CY	7.14	B5	0.060	13,939	580.8	16.91	235,665.41		0.00	6f1	2	102.16	1,424,002.25		0	1,659,668	
EM	11	Vertical Face, Forms (4 Uses)	6,272,640	SF	4.75	C5b	0.127	793,489	14,169.4	16.25	12,890,794.93	1.19	7,464,441.60	5a1	2	11.91	9,451,051.36		0	29,806,288	
EM	11	Sloped Face, Non-formed, 1' Lift	0	CY	0.00	B5	0.042	0	0.0	16.91	0.00		0.00	6f1	2	102.16	0.00		0	0	
EM	11	Roller Compacted Concrete, 1.5"-2" Ag	1,684,320	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	75,794,400				0.00		0	75,794,400	100 lbs of cement by volume
EM	11	Dump Truck (18 CY) Conveying Materi	1,684,320	CY	1.24	C1	0.012	20,212	2,526.5	11.88	240,116.66		0.00	3e1	1	91.31	1,845,620.99		0	2,085,738	(15 Min. Cycles)
EM	11	Truck Mtd. Hydraulic Crane 100 Ton, C	1,684,320	CY	0.59	D18a	0.025	42,108	1,052.7	17.43	734,110.87		0.00	8g	4	6.32	265,951.43		0	1,000,062	
EM	11	Surface Prep. Vacuum Truck	*33,802,560*	SY	0.18	C6a-	0.006	202,815	10,140.8	17.72	3,593,482.55		0.00	3j2	1	12.86	2,607,461.48		0	6,200,944	
EM	11	Surface Prep. Water Blast	33,802,560	SY	0.87	C1a	0.030	1,014,077	42,253.2	15.63	15,853,400.64		0.00	3j4	5	13.25	13,435,301.39		0	29,288,702	$87.51
EM	**11**	**Concrete Batch Plant and Delivery**	**1,734,850**	**CY**	**26.15**	**C6a**	**0.480**	**832,728**	**13,011.4**	**14.44**	**12,020,425.91**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**33,340,621.88**		**0**	**45,361,048**	$114.44
03400		**Precast Concrete**																			
03500		**Cementitious Decks and Underlay**																			
03600		**Grouts**																			
03900		**Concrete Restorations and Cleaning**																			
			0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0	***192,752,788***
		Subtotal Concrete						3,072,337			$48,167,967		$89,366,825				$65,758,595	$0	$0	$203,293,387	
		Construction Subtotal (Direct Costs)						3,655,025			$57,547,327		$105,033,576				$98,567,095	$97,562,784	$53,036,391	**$411,747,173**	

Indirect Costs

Sales Tax	6%	of purchased materials + Rental Equipment	12,216,040
Overhead and Profit	22%	of construction cost + general requirements	90,736,963
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	18,014,506
Project Reserve	5%	of construction cost	26,635,734
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	167,805,125
Construction Subtotal Indirects			**$315,408,369**
Total Construction (Directs and Indirect Costs)			**$727,155,541**
Permits			0
Design	0%	of construction cost	0
Construction Management	0%	of construction cost	0
Total			**$727,155,541**

TABLE 8.13-5

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 10a

Case: 10a

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No.: 141731.0910
Revision No.: 1
Date: 07/23/05

Opinion Of Probable Cost

EAA Reservoir A-1, Concrete Faced Rockfill Embankment (OG+23 and 3 Foot Perimeter Wall), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment & Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				58,199	990,131	0	6,089,526	0	0	7,079,657	0.8707	6,164,533	13,244,190
	Subtotal				58,199	$990,131	$0	$6,089,526	$0	$0	$7,079,657		$6,164,533	$13,244,190
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				184,168	3,024,397	2,297,434	13,031,974	0	0	18,353,805	0.8707	15,981,372	34,335,177
	Subtotal				184,168	$3,024,397	$2,297,434	$13,031,974	$0	$0	$18,353,805		$15,981,372	$34,335,177
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.8707	3,141,719	6,749,825
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,141,719	$6,749,825
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				678,114	11,118,152	9,220,584	36,287,618	0	0	56,626,354	0.8707	49,306,771	105,933,125
	Subtotal				678,114	$11,118,152	$9,220,584	$36,287,618	$0	$0	$56,626,354		$49,306,771	$105,933,125
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				85,146	1,398,277	0	1,634,970	0	6,993,089	10,026,336	0.8707	8,730,321	18,756,657
	Subtotal				85,146	$1,398,277	$0	$1,634,970	$0	$6,993,089	$10,026,336		$8,730,321	$18,756,657
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				101,957	1,546,365	9,929,920	3,114,250	96,232,382	0	110,822,917	0.8707	96,497,829	207,320,746
	Subtotal				101,957	$1,546,365	$9,929,920	$3,114,250	$96,232,382	$0	$110,822,917		$96,497,829	$207,320,746
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				20,263	305,583	0	1,335,165	1,215,766	0	2,856,514	0.8707	2,487,278	5,343,792
	Subtotal				20,263	$305,583	$0	$1,335,165	$1,215,766	$0	$2,856,514		$2,487,278	$5,343,792
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				336,006	5,145,342	2,053,202	20,951,150	0	1,060,889	29,210,583	0.8707	25,434,792	54,645,376
	Subtotal				336,006	$5,145,342	$2,053,202	$20,951,150	$0	$1,060,889	$29,210,583		$25,434,792	$54,645,376
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				25,469	388,537	143,163	1,167,477	0	0	1,699,177	0.8707	1,479,540	3,178,717
	Subtotal				25,469	$388,537	$143,163	$1,167,477	$0	$0	$1,699,177		$1,479,540	$3,178,717
11	**Embankment Construction (Cutoff Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				652,309	13,271,295	49,350,182	9,120,759	0	0	71,742,236	0.8707	62,468,758	134,210,995
	Subtotal				652,309	$13,271,295	$49,350,182	$9,120,759	$0	$0	$71,742,236		$62,468,758	$134,210,995
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.8707	145,882	313,420
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$145,882	$313,420
	Total				**2,176,014**	**$37,681,939**	**$72,994,485**	**$95,911,425**	**$97,448,148**	**$8,157,227**	**$312,193,224**		**$271,838,795**	**$584,032,019**

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 1
Date: 07/23/05

Case: 10a
Opinion Of Probable Cost

EAA Reservoir A-1, Concrete Faced Rockfill Embankment (OG+23 and 3 Foot Perimeter Wall), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment & Canal Redesign Adjustment

With Seepage Control 12.95 Miles = 68,376 Total Linear Feet
Without Seepage Control (South Side) 8.75 Miles = 46,200 Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	Labor: M-H per Unit	Labor: Man-Hours	Labor: Duration Days	Labor: Average Wage Rate	Labor: Labor Cost	Material: Unit Cost	Material: Material Cost	Equipment: Code	Equipment: No.	Equipment: Avg. Cost ($/hr)	Equipment: Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1		**General Requirements**																			
		Mobilization	1	LS	3,121,932						0.00		0.00						3,121,932	3,121,932	
		Supervision	1	LS	15,609,661						0.00		0.00						15,609,661	15,609,661	
		Temporary Construction Facilities	1	LS	7,804,831						0.00		0.00						7,804,831	7,804,831	
		Temporary Utilities	1	LS	4,682,898						0.00		0.00						4,682,898	4,682,898	
		Safety	1	LS	7,804,831						0.00		0.00						7,804,831	7,804,831	
		Miscellaneous	1	LS	6,243,864						0.00		0.00						6,243,864	6,243,864	
		Subtotal Mobilization									$0		$0				$0	$0	$45,268,017	$45,268,017	
2		**Site Work**																			
02225		**Demolition**																			
02230		**Site Clearing**																			
SC	1	Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
SC	1	Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439	
EM	1	Scraper (Strip Peat Embankment A	683760	CY	1.03	B5b	0.006	4,103	205.1	17.17	70,424.54	0.00	0.00	1s1	2	154.87	635,373.07		0	705,798	
EM	1	Surface Prep. (Strip Peat Embankn	1025640	SY	0.35	B5	0.008	8,205	683.8	17.22	141,319.52		0.00	3j4+	2	26	216,255.64		0	357,575	
EM	1 W/O	Scraper (Strip Peat Embankment A	462000	CY	1.03	B5b	0.006	2,772	138.6	17.17	47,584.15	0.00	0.00	1s1	2	154.87	429,306.13		0	476,890	
EM	1 W/O	Surface Prep. (Strip Peat Embankn	693000	SY	0.35	B5	0.008	5,544	462.0	17.22	95,486.16		0.00	3j4+	2	26	146,118.68		0	241,605	
EM	1	Scraper (Strip Peat Inside Bench A	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
EM	1 W/O	Scraper (Strip Peat Inside Bench A	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504	
EM	1	Dozer Angle Blade (Strip Peat Insic	469551	CY	1.15	B2	0.015	7,043	440	16.37	115,298	0.00	0.00	6c	1	120.02	422,678.82		0	537,977	
EM	1 W/O	Dozer Angle Blade (Strip Peat Insic	343552	CY	1.15	B2	0.015	5,153	322	16.37	84,359	0.00	0.00	6c	1	120.02	309,257.45		0	393,617	$1.18
02240		**Dewatering**																			
		Seepage Canal																			
SC	2	Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507	
SC	2	12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
		Production Blast																			
SC	4 w/o	Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356	
SC	4 w/o	12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
		Cut Off Wall																			
SC	2 w/o	Pump,12" Suction (Make-up)	31508	LF	68.94	B4	0.040	1,260	157.5	16.87	21,258.71	0.00	0.00			68.27	2,151,064.86		0	2,172,324	
SC	2 w/o	12" Dia Pipe 2,500 GPM	54722	LF	5.48	B4	0.098	5,363	670.3	16.87	90,456.10	0.00	0.00			3.83	209,584.82		0	300,041	
02300		**Earthwork**																			
02305		**Equipment Mobilization Or Demobilization**																			
	12	Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049	Other: Driving Cost for 140Miles
	12	Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417	Other: Driving Cost for 140Miles
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12	Crawler Type Drill, 4"	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736	Other: Driving Cost for 140Miles
	12	Air Compressor, 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	Excavator, Diesel Hydraulic, Crawle	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	**Concrete Batch Plant, Portable, 2**	**8**	**EA**	**3,466**	**C1**	16.000	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Wheeled Skid Steer, Diesel, w/ Bro	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310		**Grading**																			
02315		**Excavation and Fill**			13																
SC	2	Drilling and Blasting (Seepage Can	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2	Excavating Cap Rock (Seepage Ca	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2	Dump Truck (Canal/Stock Pile Area	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2	Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2	Dozer Angle Blade (Stock Pile Area	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
					13.32																***18,353,805***
SC	3	Excavated - Silty, Sand, Shells (Se	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3	Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812	
SC	3	Haul to Dewater and Work Stock Pi	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719	(Avg. 1,000 FT. Around Trip)
SC	3	Dozer Angle Blade - Work Stock Pi	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790	***3,608,106***
EM	4	Drilling and Blasting (Prod. Blast Ar	2,817,305	CY	6.02	B6	0.080	225,384	9,391.0	17.11	3,857,078.27	1.89	5,324,706.33	6h	4	34.56	7,790,294.27		0	16,972,079	
EM	4	Excavating Cap Rock (Prod. Blast /	2,817,305	CY	1.75	B2	0.015	41,661	2,603.8	16.37	681,988.88	0.00	0.00	1b	1	102.03	4,250,791.02		0	4,932,780	
EM	4	Dump Truck (Prod. Blast Stock Pile	3,521,631	CY	1.71	C1	0.017	58,358	7,294.8	11.88	693,298.50	0.00	0.00	3e1	1	91.31	5,328,935.79		0	6,022,234	(Avg. 2 Miles Around Trip)
EM	4	Hoe Ram (Stock Pile Area)	369,771	CY	1.74	A10	0.077	28,472	2,847.2	17.89	509,371.02	0.00	0.00	6i	1	5.91	134,663.01		0	644,034	
EM	4	Dozer Angle Blade (Prod. Blast Are	3,521,631	CY	1.20	B2	0.016	55,466	3,466.6	16.37	907,973.36	0.00	0.00	6c	1	120.02	3,328,595.72		0	4,236,569	32,807,696

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 1
Date: 07/23/05

Case: 10a
Opinion Of Probable Cost

EAA Reservoir A-1, Concrete Faced Rockfill Embankment (OG+23 and 3 Foot Perimeter Wall), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment & Canal Redesign Adjustment

With Seepage Control 12.95 Miles = 68,376 Total Linear Feet
Without Seepage Control (South Side) 8.75 Miles = 46,200 Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	4 Excavated - Silty, Sand, Shells (Prc	474,929	CY	0.80	B3	0.005	2,375	98.9	16.75	39,775.27	0.00	0.00	1n	1	144.10	342,193.49		0	381,969	
EM	4 Haul to Dewater and Work Stock Pi	522,421	CY	1.71	C1	0.017	8,657	1,082.2	11.88	102,848.36	0.00	0.00	3e1	1	91.31	790,528.59		0	893,377	(Avg. 2 Miles Around Trip)
EM	4 Dozer Angle Blade (Work Stock Pil	522,421	CY	0.69	B2	0.009	4,702	293.9	16.37	76,968.35	0.00	0.00	6c	1	120.02	282,163.02		0	359,131	1,634,477
																				36,722,106
EM	4 W/O Drilling and Blasting (Prod. Blast Ar	2,061,311	CY	6.02	B6	0.080	164,905	6,871.0	17.11	2,822,072.28	1.89	3,895,877.92	6h	4	34.56	5,699,851.52		0	12,417,802	
EM	4 W/O Excavating Cap Rock (Prod. Blast /	2,061,311	CY	1.75	B2	0.015	30,482	1,905.1	16.37	498,984.41	0.00	0.00	1b	1	102.03	3,110,136.39		0	3,609,121	
EM	4 W/O Dozer Angle Blade (Prod. Blast Are	2,473,573	CY	1.20	B2	0.016	38,959	2,434.9	16.37	637,755.22	0.00	0.00	6c	1	120.02	2,337,986.31		0	2,975,742	19,002,664
																				65,388,369
EM	4 W/O Excavated - Silty, Sand, Shells (Prc	261,972	CY	0.80	B3	0.005	1,310	54.6	16.75	21,940.18	0.00	0.00	1n	1	144.10	188,755.19		0	210,695	
EM	4 W/O Haul to Dewater and Work Stock Pi	288,170	CY	1.71	C1	0.017	4,775	596.9	11.88	56,731.53	0.00	0.00	3e1	1	91.31	436,058.47		0	492,790	(Avg. 2 Miles Around Trip)
EM	4 W/O Dozer Angle Blade (Work Stock Pil	288,170	CY	0.69	B2	0.009	2,594	162.1	16.37	42,456.02	0.00	0.00	6c	1	120.02	155,642.16		0	198,098	901,584
EM	5 Crusher (Level Coarse Borrow Are:	186,759	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,389,674	3,389,674	
EM	5 Dozer Loader (Level Coarse Borrov	186,759	CY	0.82	B5	0.013	2,381	198.4	17.22	41,011.78	0.00	0.00	6j	1	47.46	113,002.50		0	154,014	
EM	5 Dump Truck (Level Coarse Borrow	261,462	CY	1.71	C1	0.017	4,333	541.6	11.88	51,473.74	0.00	0.00	3e1	1	91.31	395,645.21		0	447,119	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankm	261,462	CY	1.03	B2	0.014	3,530	220.6	16.37	57,781.89	0.00	0.00	6c	1	120.02	211,826.21		0	269,608	4,260,415
EM	5 Crusher (Granular Toe Borrow Are:	194,779	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,603,415	3,603,415	
EM	5 Dozer Loader (Granular Toe Borrov	194,779	CY	0.82	B5	0.013	2,483	207.0	17.22	42,773.03	0.00	0.00	6j	1	47.46	117,855.37		0	160,628	
EM	5 Dump Truck (Granular Toe Borrow	272,691	CY	1.71	C1	0.017	4,519	564.9	11.88	53,684.26	0.00	0.00	3e1	1	91.31	412,636.11		0	466,320	(Avg. 2 Miles Around Trip)
EM	5 Backhoe Loader (New Embankmer	272,691	CY	3.84	B4-	0.160	43,631	1,817.9	16.65	726,593.94	0.00	0.00	2a	1	7.33	319,874.67		0	1,046,469	
EM	5 Compacting (New Embankment Ar	272,691	CY	1.79	A1	0.089	24,269	3,033.7	17.51	424,958.74	0.00	0.00	8b2	1	2.64	64,129.59		0	489,088	5,765,921
																				5,765,921
EM	6 Clean Cap Rock Surface (Embankr	183,322	SY	0.87	C1a	0.030	5,500	229.2	15.63	85,977.83		0.00	3j4	5	13.25	72,863.74		0	158,842	
EM	6 Cement Grout Cap Rock (Embankr	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 w/o Cut Off Wall (Embankment Area)	114,576	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	9,166,080	0	9,166,080	(Quoted for 2' Wide at $40/LF)
EM	6 w/o Lean Concrete Fill In Cap Rock Cu	152,768	CY	67.54	D8	0.137	20,929	436.0	17.69	370,133.18	65.00	9,929,920.00	8b1	2	0.83	17,387.96		0	10,317,441	
EM	6 w/o Concrete Batch Plant and Delive	157,351	CY	26.15	C6a	0.480	75,528	1,180.1	14.44	1,090,253.89	0.00	0.00	8h1	6	40.04	3,023,997.89		0	4,114,252	
EM	6 w/o Cut Off Wall (Embankment Area)	3,437,280	SF	25.00	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	85,932,000	0	85,932,000	(Plastic Concrete)
SC	6 w/o Cut Off Wall (Through Limestone)	25,207	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	1,134,302	0	1,134,302	110,822,917
EM	7 Dozer Loader (Sand Horiz. Filter)	309,355	CY	0.46	B2	0.006	1,856	116.0	16.37	30,384.87	0.00	0.00	6c	1	120.02	111,389.77		0	141,775	
EM	7 Dump Truck (Sand Horiz. Filter)	309,355	CY	0.93	C1	0.009	2,784	348.0	11.88	33,076.26	0.00	0.00	3e1	1	91.31	254,235.74		0	287,312	
EM	7 Wash and Screen (Sand Horiz. Filt	309,355	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	1,215,766	0	1,215,766	
EM	7 Dozer Angle Blade (Sand Horiz. Fil	309,355	CY	0.69	B2	0.009	2,784	174.0	16.37	45,577.30	0.00	0.00	6c	1	120.02	167,084.65		0	212,662	
EM	7 Dozer and Dump Trucks (Sand Hor	309,355	CY	1.75	C6	0.024	7,270	259.6	14.17	103,013.73	0.00	0.00	3f1	2	60.38	438,968.84		0	541,983	
EM	7 Dozer Angle Blade (Sand Horiz. Fil	309,355	CY	0.69	B2	0.009	2,784	174.0	16.37	45,577.30	0.00	0.00	6c	1	120.02	167,084.65		0	212,662	
EM	7 Compact (Sand Horiz. Filter)	309,355	CY	0.79	B5	0.009	2,784	232.0	17.22	47,953.15	0.00	0.00	6g1	1	70.54	196,401.86		0	244,355	
EM	8 Dozer and Dump Trucks (Mass Rai	1,879,046	CY	1.75	C6	0.024	44,158	1,577.1	14.17	625,713.06	0.00	0.00	3f1	2	60.38	2,666,329.24		0	3,292,042	
EM	8 Dozer Angle Blade (Mass Random	1,879,046	CY	0.69	B2	0.009	16,911	1,057.0	16.37	276,839.91	0.00	0.00	6c	1	120.02	1,014,884.54		0	1,291,724	
EM	8 Compact (Mass Random Fill)	1,879,046	CY	0.79	B5	0.009	16,911	1,409.3	17.22	291,270.98	0.00	0.00	6g1	1	70.54	1,192,959.42		0	1,484,230	6,067,997
EM	8 Dozer Loader (Rock Fill Borrow Are	4,926,768	CY	1.15	B2	0.015	73,902	4,618.8	16.37	1,209,767.88	0.00	0.00	6c	1	120.02	4,434,962.93		0	5,644,731	
EM	8 Dump Truck (Rock Fill Borrow Area	6,897,475	CY	0.93	C1	0.009	62,077	7,759.7	11.88	737,478.05	0.00	0.00	3e1	1	91.31	5,668,515.34		0	6,405,993	
EM	8 Dozer Angle Blade (New Embankm	6,897,475	CY	1.03	B2	0.014	93,116	5,819.7	16.37	1,524,307.53	0.00	0.00	6c	1	120.02	5,588,053.29		0	7,112,361	19,163,085
EM	10 Common Borrow (Top Soil - Peat)	190,085	CY	3.65	C6	0.047	8,934	319.1	14.17	126,594.90	0.00	0.00	3f	3	63.57	567,947.66		0	694,543	
EM	10 W/O Common Borrow (Top Soil - Peat)	128,436	CY	3.65	C6	0.047	6,036	215.6	14.17	85,537.09	0.00	0.00	3f	3	63.57	383,748.42		0	469,286	1,163,828
02370	**Erosion and Sedimentation Control**																			
02600	**Drainage and Containment**																			
02620	**Subdrainage System (Seepage Water)**																			
EM	8 Geomembrane Placement	1,604,064	SY	1.64	B4-	0.015	24,061	1,002.5	16.65	400,695.19	1.28	2,053,201.92	2a	1	7.33	176,401.47		0	2,630,299	
02700	**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o Crusher (Transition Borrow Area)	84871	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,060,889	1,060,889	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" T	254613	SY	1.13	B7a	0.019	4,869	67.6	16.28	79,269.72	0.00	0.00	7a+	7	42.93	209,044.09		0	288,314	3,979,501
02910	**Plant Preparation**																			
EM	10 Fine Grading	569,572	SY	0.32	B5a	0.008	4,557	284.8	17.30	78,805.99	0.00	0.00	7d	1	22.97	104,665.88		0	183,472	
EM	10 W/O Fine Grading	384,846	SY	0.32	B5a	0.008	3,079	192.4	17.30	53,247.29	0.00	0.00	7d	1	22.97	70,720.19		0	123,967	
02920	**Lawns and Grasses**																			
EM	10 Hydro or Air Seeding w/ Mulch and	569,572	SY	0.24	C4a	0.003	1,709	3.0	15.49	26,468.01	0.15	85,435.81	3j5	2	14.11	24,106.84		0	136,011	443,450
EM	10 W/O Hydro or Air Seeding w/ Mulch and	384,846	SY	0.24	C4a	0.003	1,155	3.0	15.49	17,883.79	0.15	57,726.90	3j5	2	14.11	16,288.40		0	91,899	535,349
	Subtotal Site Construction						1,523,705			$24,410,643		$23,644,302				$86,790,666	$97,448,148	$8,157,227	$240,450,988	

EAA Reservoir A-1 Basis of Design Report

Case: 10a

Opinion Of Probable Cost

January, 2006

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 1
Date: 07/23/05

EAA Reservoir A-1, Concrete Faced Rockfill Embankment (OG+23 and 3 Foot Perimeter Wall), With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment & Canal Redesign Adjustment

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

						Labor						Material		Equipment							
CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
3		**Concrete**																			
03050		**Basic Concrete Materials and Methods**																			
03100		**Concrete and Forms and Accessories**																			
EM	11	Edge Forms, Wood, 4 use, 10" Higl	95,480	SFCA	2.43	D3	0.074	7,066	220.8	22.87	161,553.11	0.74	70,655.20				0.00		0	232,208	
EM	11	Waterstop PVC,Ribbed 3/16" Thick	343,728	LF	2.60	D3	0.055	18,905	590.8	22.87	432,263.74	1.34	460,595.52				0.00		0	892,859	
03200		**Concrete Reinforcement**																			
EM	11	In Place, #7 @ 12" EW	11,453	TN	1,209.62	D10d	13.913	159,346	4,979.6	30.52	4,863,234.58	785.00	8,990,618.31				0.00		0	13,853,853	
EM	11	Unloading and Sorting	11,453	TN	20.06	D11d	0.560	6,414	114.5	26.76	171,648.66		0.00	5a	1	63.47	**58,149.83**		0	229,798	
EM	11	Crane Handling	11,453	TN	14.87	D11d	0.415	4,753	84.9	26.76	127,203.91		0.00	5a	1	63.47	**43,093.18**		0	170,297	1,245
03300		**Cast-In-Place Concrete**																			
EM	11	Concrete Face Slab	155,823	CY	61.03	D8b	0.346	53,915	842.4	17.59	948,228.00	51.85	8,079,441.22	8b	3	8.93	481,620.50		0	9,509,290	
EM	**11**	**Concrete Batch Plant and Delive**	**160,498**	**CY**	**26.15**	**C6a**	**0.480**	**77,039**	**1,203.7**	**14.44**	**1,112,058.96**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,084,477.85**		**0**	**4,196,537**	$186.65
03310		**Structural Concrete**																			
EM	11	Cutoff Wall Cap	114,576	CY	236.50	D8b	1.382	158,344	2,474.1	17.59	2,784,875.66	199.85	22,898,013.60	8b	3	8.93	1,414,483.88		0	27,097,373	
EM	**11**	**Concrete Batch Plant and Delive**	**118,013**	**CY**	**26.15**	**C6a**	**0.480**	**56,646**	**885.1**	**14.44**	**817,690.41**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**2,267,998.42**		**0**	**3,085,689**	$263.43
EM	11	Parapet Wall Footing	34,373	CY	236.50	D8b	1.382	47,503	742.2	17.59	835,462.70	199.85	6,869,404.08	8b	3	8.93	424,345.16		0	8,129,212	
EM	11	Parapet Wall	16,957	CY	174.72	D8b	2.182	37,001	578.1	17.59	650,750.08	116.85	1,981,454.43	8b	3	8.93	330,526.60		0	2,962,731	
EM	**11**	**Concrete Batch Plant and Delive**	**52,870**	**CY**	**26.15**	**C6a**	**0.480**	**25,378**	**396.5**	**14.44**	**366,325.31**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,016,063.29**		**0**	**1,382,389**	$243.02 **#DIV/0!**
		Subtotal Concrete						652,309			$13,271,295		$49,350,182				$9,120,759	$0	$0	$71,742,236	
		Construction Subtotal (Direct Costs)						2,176,014			$37,681,939		$72,994,485				$95,911,425	$97,448,148	$53,425,245	**$357,461,241**	

Indirect Costs			
Sales Tax	6%	of purchased materials + Rental Equipment	10,134,355
Overhead and Profit	13%	of construction cost + general requirements	45,797,916
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	14,468,773
Project Reserve	5%	of construction cost	21,393,114
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	134,776,620
Construction Subtotal Indirects			**$226,570,778**
Total Construction (Directs and Indirect Costs)			**$584,032,019**
Permits			0
Design	0%	of construction cost	0
Construction Management	0%	of construction cost	0
Total			**$584,032,019**

TABLE 8.13-6

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 6

Case: 6

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02

Project No.: 141731.0910

Revision No.: 2

Date: 07/27/05

Opinion Of Probable Cost

Reservoir A-1, U/S Rockfill Embankment (OG +24) "Incl. 15 Foot Wave Break Bench OG +16," With 30' Cutoff Wall, and Seepage Canal 15 Feet Deep

South Side Embankment and Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				62,342	1,063,096	0	6,394,844	0	0	7,457,940	0.9505	7,088,620	14,546,560
	Subtotal				62,342	$1,063,096	$0	$6,394,844	$0	$0	$7,457,940		$7,088,620	$14,546,560
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				184,168	3,024,397	2,297,434	13,031,974	0	0	18,353,805	0.9505	17,444,918	35,798,723
	Subtotal				184,168	$3,024,397	$2,297,434	$13,031,974	$0	$0	$18,353,805		$17,444,918	$35,798,723
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.9505	3,429,432	7,037,538
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,429,432	$7,037,538
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				785,657	12,346,851	7,682,910	49,457,523	0	0	69,487,283	0.9505	66,046,248	135,533,531
	Subtotal				785,657	$12,346,851	$7,682,910	$49,457,523	$0	$0	$69,487,283		$66,046,248	$135,533,531
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				58,552	940,651	0	1,734,007	0	7,775,288	10,449,947	0.9505	9,932,462	20,382,409
	Subtotal				58,552	$940,651	$0	$1,734,007	$0	$7,775,288	$10,449,947		$9,932,462	$20,382,409
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				78,671	1,201,496	5,925,920	2,094,665	11,275,202	0	20,497,283	0.9505	19,482,250	39,979,533
	Subtotal				78,671	$1,201,496	$5,925,920	$2,094,665	$11,275,202	$0	$20,497,283		$19,482,250	$39,979,533
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				33,144	527,689	0	1,231,846	1,168,922	0	2,928,457	0.9505	2,783,439	5,711,895
	Subtotal				33,144	$527,689	$0	$1,231,846	$1,168,922	$0	$2,928,457		$2,783,439	$5,711,895
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				675,707	10,520,940	8,421,336	40,120,452	0	822,189	59,884,917	0.9505	56,919,395	116,804,312
	Subtotal				675,707	$10,520,940	$8,421,336	$40,120,452	$0	$822,189	$59,884,917		$56,919,395	$116,804,312
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				22,282	340,602	128,812	1,010,128	0	0	1,479,542	0.9505	1,406,275	2,885,817
	Subtotal				22,282	$340,602	$128,812	$1,010,128	$0	$0	$1,479,542		$1,406,275	$2,885,817
11	**Embankment Construction (Cutoff Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				256,221	3,740,674	20,572,121	11,276,428	0	0	35,589,222	0.9505	33,826,831	69,416,054
	Subtotal				256,221	$3,740,674	$20,572,121	$11,276,428	$0	$0	$35,589,222		$33,826,831	$69,416,054
12	**Equipment Mobilization or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.9505	159,241	326,779
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$159,241	$326,779
	Total				**2,191,126**	**$34,200,256**	**$45,028,532**	**$129,530,401**	**$12,444,124**	**$8,700,726**	**$229,904,040**		**$218,519,110**	**$448,423,150**

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, U/S Rockfill Embankment (OG +24) "Incl. 15 Foot Wave Break Bench OG +16," With 30' Cutoff Wall, and Seepage Canal 15 Feet De
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376 Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200 Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	Labor: M-H per Unit	Labor: Man Hours	Labor: Duration Days	Labor: Average Wage Rate	Labor: Labor Cost	Material: Unit Cost	Material: Material Cost	Equipment: Code	Equipment: No.	Equipment: Avg. Cost ($/hr)	Equipment: Equipment Cost	Sub-contract	Other	Total Cost
1		**General Requirements**																		
		Mobilization	1	LS	2,299,040						0.00		0.00						2,299,040	2,299,040
		Supervision	1	LS	11,495,202						0.00		0.00						11,495,202	11,495,202
		Temporary Construction Facilities	1	LS	5,747,601						0.00		0.00						5,747,601	5,747,601
		Temporary Utilities	1	LS	3,448,561						0.00		0.00						3,448,561	3,448,561
		Safety	1	LS	5,747,601						0.00		0.00						5,747,601	5,747,601
		Miscellaneous	1	LS	4,598,081						0.00		0.00						4,598,081	4,598,081
		Subtotal Mobilizatior									$0		$0				$0	$0	$33,336,086	$33,336,086
2		**Site Work**																		
02225		**Demolition**																		
02230		**Site Clearing**																		
SC	1	Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626
SC	1	Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439
EM	1	Scraper (Strip Peat Embankment /	1001025	CY	1.03	B5b	0.006	6,006	300.3	17.17	103,101.53	0.00	0.00	1s1	2	154.87	930,186.17		0	1,033,288
EM	1	Surface Prep. (Strip Peat Embankı	1501537	SY	0.35	B5	0.008	12,012	1,001.0	17.22	206,891.77		0.00	3j4+	2	26	316,598.26		0	523,490
EM	1 W/O	Scraper (Strip Peat Embankment /	487872	CY	1.03	B5b	0.006	2,927	146.4	17.17	50,248.86	0.00	0.00	1s1	2	154.87	453,347.27		0	503,596
EM	1 W/O	Surface Prep. (Strip Peat Embankı	731808	SY	0.35	B5	0.008	5,854	487.9	17.22	100,833.38		0.00	3j4+	2	26	154,301.32		0	255,135
EM	1	Scraper (Strip Peat Inside Bench /	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626
EM	1 W/O	Scraper (Strip Peat Inside Bench /	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504
EM	1	Dozer Angle Blade (Strip Peat Insi	551118	CY	1.15	B2	0.015	8,267	517	16.37	135,327	0.00	0.00	6c	1	120.02	496,103.88		0	631,431
EM	1 W/O	Dozer Angle Blade (Strip Peat Insi	126387	CY	1.15	B2	0.015	1,896	118	16.37	31,034	0.00	0.00	6c	1	120.02	113,770.72		0	144,805
02240		**Dewatering**																		
		Seepage Canal																		
SC	2	Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507
SC	2	12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925
		Production Blast																		
SC	4 w/o	Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356
SC	4 w/o	12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578
		Cutoff Wall																		
SC	2 w/o	Pump,12" Suction (Make-up)	31508	LF	68.94	B4	0.040	1,260	157.5	16.87	21,258.71	0.00	0.00			68.27	2,151,064.86		0	2,172,324
SC	2 w/o	12" Dia Pipe 2,500 GPM	54722	LF	5.48	B4	0.098	5,363	670.3	16.87	90,456.10	0.00	0.00			3.83	209,584.82		0	300,041
02300		**Earthwork**																		
02305		**Equipment Mobilization or Demobilizatior**																		
	12	Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049
	12	Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242
	12	Crawler Type Drill, 4'	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736
	12	Air Compressor, 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0
	12	50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0
	12	Excavator, Diesel Hydraulic, Crawl	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	**Concrete Batch Plant, Portable,**	**8**	**EA**	**3,466**	**C1**	16.000	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777
	12	Truck Mtd. Crane Over 75 Tor	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Wheeled Skid Steer, Diesel, w/ Brc	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311
	12	Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
02310		**Grading**																		
02315		**Excavation and Fil**			13															
SC	2	Drilling and Blasting (Seepage Car	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887
SC	2	Excavating Cap Rock (Seepage C	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330
SC	2	Dump Truck (Canal/Stock Pile Are	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973
SC	2	Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879
SC	2	Dozer Angle Blade (Stock Pile Are	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939
					13.32															
SC	3	Excavated - Silty, Sand, Shells (Se	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786
SC	3	Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812
SC	3	Haul to Dewater and Work Stock F	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719
SC	3	Dozer Angle Blade - Work Stock P	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790
EM	4	Drilling and Blasting (Prod. Blast A	3,306,709	CY	6.02	B6	0.080	264,537	11,022.4	17.11	4,527,105.22	1.89	6,249,680.22	6h	4	34.56	9,143,574.32		0	19,920,360
EM	4	Excavating Cap Rock (Prod. Blast	3,306,709	CY	1.75	B2	0.015	48,898	3,056.1	16.37	800,459.62	0.00	0.00	1b	1	102.03	4,989,211.22		0	5,789,671
EM	4	Dump Truck (Prod. Blast Stock Pil	4,133,386	CY	1.71	C1	0.017	68,496	8,562.0	11.88	813,733.87	0.00	0.00	3e1	1	91.31	6,254,644.40		0	7,068,378
EM	4	Hoe Ram (Stock Pile Area)	434,006	CY	1.74	A10	0.077	33,418	3,341.8	17.89	597,855.69	0.00	0.00	6i	1	5.91	158,055.80		0	755,911
EM	4	Dozer Angle Blade (Prod. Blast Ar	4,133,386	CY	1.20	B2	0.016	65,101	4,068.8	16.37	1,065,700.68	0.00	0.00	6c	1	120.02	3,906,818.06		0	4,972,519
EM	4	Excavated - Silty, Sand, Shells (Pr	4,159,601	CY	0.80	B3	0.005	20,798	866.6	16.75	348,366.57	0.00	0.00	1n	1	144.10	2,997,057.85		0	3,345,424
EM	4	Haul to Dewater and Work Stock F	4,575,561	CY	1.71	C1	0.017	75,824	9,477.9	11.88	900,784.15	0.00	0.00	3e1	1	91.31	6,923,743.37		0	7,824,528
EM	4	Dozer Angle Blade (Work Stock Pi	4,575,561	CY	0.69	B2	0.009	41,180	2,573.8	16.37	674,117.39	0.00	0.00	6c	1	120.02	2,471,288.66		0	3,145,406

EAA Reservoir A-1 Basis of Design Report

Case: 6

Opinion Of Probable Cost

January, 2006

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, U/S Rockfill Embankment (OG +24) "Incl. 15 Foot Wave Break Bench OG +16," With 30' Cutoff Wall, and Seepage Canal 15 Feet De
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost
EM	4 W/O Drilling and Blasting (Prod. Blast A	758,322	CY	6.02	B6	0.080	60,666	2,527.7	17.11	1,038,193.92	1.89	1,433,229.34	6h	4	34.56	2,096,881.52		0	4,568,305
EM	4 W/O Excavating Cap Rock (Prod. Blast	758,322	CY	1.75	B2	0.015	11,214	700.9	16.37	183,568.15	0.00	0.00	1b	1	102.03	1,144,167.96		0	1,327,736
EM	4 W/O Dozer Angle Blade (Prod. Blast Ar	909,987	CY	1.20	B2	0.016	14,332	895.8	16.37	234,619.64	0.00	0.00	6c	1	120.02	860,106.66		0	1,094,726
EM	4 W/O Excavated - Silty, Sand, Shells (Pr	2,148,580	CY	0.80	B3	0.005	10,743	447.6	16.75	179,943.58	0.00	0.00	1n	1	144.10	1,548,085.69		0	1,728,029
EM	4 W/O Haul to Dewater and Work Stock F	2,363,438	CY	1.71	C1	0.017	39,166	4,895.7	11.88	465,286.66	0.00	0.00	3e1	1	91.31	3,576,356.73		0	4,041,643
EM	4 W/O Dozer Angle Blade (Work Stock Pi	2,363,438	CY	0.69	B2	0.009	21,271	1,329.4	16.37	348,205.32	0.00	0.00	6c	1	120.02	1,276,507.43		0	1,624,713
EM	5 Crusher (Level Coarse Borrow Are	152,386	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,765,807	2,765,807
EM	5 Dozer Loader (Level Coarse Borro	152,386	CY	0.82	B5	0.013	1,943	161.9	17.22	33,463.60	0.00	0.00	6j	1	47.46	92,204.49		0	125,668
EM	5 Dump Truck (Level Coarse Borrow	213,341	CY	1.71	C1	0.017	3,535	441.9	11.88	42,000.04	0.00	0.00	3e1	1	91.31	322,827.08		0	364,827
EM	5 Dozer Angle Blade (New Embankr	213,341	CY	1.03	B2	0.014	2,880	180.0	16.37	47,147.19	0.00	0.00	6c	1	120.02	172,839.79		0	219,987
EM	5 Crusher (Bedding Borrow Area)	167,281	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,036,149	3,036,149
EM	5 Dozer Loader (Bedding Borrow Ar	167,281	CY	0.82	B5	0.013	2,133	177.7	17.22	36,734.48	0.00	0.00	6j	1	47.46	101,216.96		0	137,951
EM	5 Dump Truck (Bedding Borrow Area	234,193	CY	1.71	C1	0.017	3,881	485.1	11.88	46,105.31	0.00	0.00	3e1	1	91.31	354,381.60		0	400,487
EM	5 Dozer Angle Blade (New Embankr	234,193	CY	1.03	B2	0.014	3,162	197.6	16.37	51,755.56	0.00	0.00	6c	1	120.02	189,733.90		0	241,489
EM	5 Crusher (Granular Toe Borrow Are	106,667	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,973,331	1,973,331
EM	5 Dozer Loader (Granular Toe Borro	106,667	CY	0.82	B5	0.013	1,360	113.3	17.22	23,423.71	0.00	0.00	6j	1	47.46	64,540.91		0	87,965
EM	5 Dump Truck (Granular Toe Borrow	149,333	CY	1.71	C1	0.017	2,475	309.3	11.88	29,399.01	0.00	0.00	3e1	1	91.31	225,971.12		0	255,370
EM	5 Backhoe Loader (New Embankme	149,333	CY	3.84	B4-	0.160	23,893	995.6	16.65	397,903.25	0.00	0.00	2a	1	7.33	175,172.35		0	573,076
EM	5 Compacting (New Embankment Ar	149,333	CY	1.79	A1	0.089	13,291	1,661.3	17.51	232,719.34	0.00	0.00	8b2	1	2.64	35,119.16		0	267,839
EM	6 Clean Cap Rock Surface (Embank	380,171	SY	0.87	C1a	0.030	11,405	475.2	15.63	178,299.99		0.00	3j4	5	13.25	151,104.12		0	329,404
EM	6 Cement Grout Cap Rock (Embank	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0
EM	6 W/O Clean Cap Rock Surface (Embank	323,400	SY	0.87	C1a	0.030	9,702	404.3	15.63	151,674.60		0.00	3j4	5	13.25	128,539.86		0	280,214
EM	6 W/O Cement Grout Cap Rock (Embank	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0
EM	6 w/o Cut Off Wall (Embankment Area)	68,376	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	5,470,080	0	5,470,080
EM	6 w/o Lean Concrete Fill In Cap Rock Cu	91,168	CY	67.54	D8	0.137	12,490	260.2	17.69	220,885.93	65.00	5,925,920.00	8b1	2	0.83	10,376.69		0	6,157,183
EM	6 w/o **Concrete Batch Plant and Delive**	**93,903**	**CY**	**26.15**	**C6a**	**0.480**	**45,073**	**704.3**	**14.44**	**650,635.38**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,804,643.90**		**0**	**2,455,279**
EM	6 w/o Cut Off Wall (Embankment Area)	2,051,280	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	5,128,200	0	5,128,200
SC	6 w/o Cut Off Wall (Through Limestone)	15,043	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	676,922	0	676,922
EM	7 Dozer Loader (Sand Horiz. Filter)	196,239	CY	0.46	B2	0.006	1,177	73.6	16.37	19,274.61	0.00	0.00	6c	1	120.02	70,659.97		0	89,935
EM	7 Dump Truck (Sand Horiz. Filter)	196,239	CY	0.93	C1	0.009	1,766	220.8	11.88	20,981.89	0.00	0.00	3e1	1	91.31	161,274.15		0	182,256
EM	7 Wash and Screen (Sand Horiz. Filt	196,239	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	771,220	0	771,220
EM	7 Dozer Angle Blade (Sand Horiz. Fi	196,239	CY	0.69	B2	0.009	1,766	110.4	16.37	28,911.91	0.00	0.00	6c	1	120.02	105,989.96		0	134,902
EM	7 Dozer and Dump Trucks (Sand Ho	196,239	CY	1.75	C6	0.024	4,612	164.7	14.17	65,346.65	0.00	0.00	3f1	2	60.38	278,459.38		0	343,806
EM	7 Dozer Angle Blade (Sand Horiz. Fi	196,239	CY	0.69	B2	0.009	1,766	110.4	16.37	28,911.91	0.00	0.00	6c	1	120.02	105,989.96		0	134,902
EM	7 Compact (Sand Horiz. Filter)	196,239	CY	0.79	B5	0.009	1,766	147.2	17.22	30,419.03	0.00	0.00	6g1	1	70.54	124,587.29		0	155,006
EM	7 Dozer Loader (Sand Vert. Filter)	101,196	CY	0.46	B2	0.006	607	37.9	16.37	9,939.52	0.00	0.00	6c	1	120.02	36,437.89		0	46,377
EM	7 Dump Truck (Sand Vert. Filter)	101,196	CY	0.93	C1	0.009	911	113.8	11.88	10,819.93	0.00	0.00	3e1	1	91.31	83,165.76		0	93,986
EM	7 Wash and Screen (Sand Vert. Filte	101,196	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	397,702	0	397,702
EM	7 Dozer Angle Blade (Sand Vert. Filt	101,196	CY	0.69	B2	0.009	911	56.9	16.37	14,909.28	0.00	0.00	6c	1	120.02	54,656.84		0	69,566
EM	7 Dozer and Dump Trucks (Sand Ve	101,196	CY	1.75	C6	0.024	2,378	84.9	14.17	33,697.92	0.00	0.00	3f1	2	60.38	143,595.78		0	177,294
EM	7 Backhoe (Sand Vert. Filter)	101,196	CY	3.28	B4a	0.153	15,483	322.6	17.08	264,476.49	0.00	0.00	2	1	4.33	67,028.75		0	331,505
EM	8 Dozer and Dump Trucks (Max. 6" I	1,843,417	CY	1.75	C6	0.024	43,320	1,547.2	14.17	613,848.63	0.00	0.00	3f1	2	60.38	2,615,771.77		0	3,229,620
EM	8 Dozer With Ripper Attach, (Max. 6	1,843,417	CY	2.04	B5	0.020	36,868	3,072.4	17.22	634,995.70	0.00	0.00	6g	2	84.81	3,126,873.96		0	3,761,870
EM	8 Dozer Angle Blade (Max. 6" Rando	1,843,417	CY	0.69	B2	0.009	16,591	1,036.9	16.37	271,590.62	0.00	0.00	6c	1	120.02	995,640.86		0	1,267,231
EM	8 Compact (Max. 6" Random Fill)	1,843,417	CY	0.79	B5	0.009	16,591	1,382.6	17.22	285,748.06	0.00	0.00	6g1	1	70.54	1,170,339.18		0	1,456,087
EM	8 W/O Dozer and Dump Trucks (Max. 6" I	1,601,754	CY	1.75	C6	0.024	37,641	1,344.3	14.17	533,376.07	0.00	0.00	3f1	2	60.38	2,272,856.87		0	2,806,233
EM	8 W/O Dozer With Ripper Attach, (Max. 6	1,601,754	CY	2.04	B5	0.020	32,035	2,669.6	17.22	551,750.86	0.00	0.00	6g	2	84.81	2,716,956.05		0	3,268,707
EM	8 W/O Dozer Angle Blade (Max. 6" Rando	1,601,754	CY	0.69	B2	0.009	14,416	901.0	16.37	235,986.42	0.00	0.00	6c	1	120.02	865,117.21		0	1,101,104
EM	8 W/O Compact (Max. 6" Random Fill)	1,601,754	CY	0.79	B5	0.009	14,416	1,201.3	17.22	248,287.89	0.00	0.00	6g1	1	70.54	1,016,913.43		0	1,265,201
EM	8 Dozer and Dump Trucks (Mass Ra	2,916,236	CY	1.75	C6	0.024	68,532	2,447.6	14.17	971,092.14	0.00	0.00	3f1	2	60.38	4,138,081.09		0	5,109,173
EM	8 Dozer Angle Blade (Mass Random	2,916,236	CY	0.69	B2	0.009	26,246	1,640.4	16.37	429,649.11	0.00	0.00	6c	1	120.02	1,575,077.25		0	2,004,726
EM	8 Compact (Mass Random Fill)	2,916,236	CY	0.79	B5	0.009	26,246	2,187.2	17.22	452,045.80	0.00	0.00	6g1	1	70.54	1,851,445.33		0	2,303,491
EM	8 W/O Dozer and Dump Trucks (Mass Ra	*1,777,314*	CY	1.75	C6	0.024	41,767	1,491.7	14.17	591,836.68	0.00	0.00	3f1	2	60.38	2,521,973.00		0	3,113,810
EM	8 W/O Dozer Angle Blade (Mass Random	1,777,314	CY	0.69	B2	0.009	15,996	999.7	16.37	261,851.67	0.00	0.00	6c	1	120.02	959,938.24		0	1,221,790
EM	8 W/O Compact (Mass Random Fill)	1,777,314	CY	0.79	B5	0.009	15,996	1,333.0	17.22	275,501.44	0.00	0.00	6g1	1	70.54	1,128,372.07		0	1,403,874
EM	8 Dozer Loader (Rock Fill Borrow Ar	2,826,442	CY	1.15	B2	0.015	42,397	2,649.8	16.37	694,032.85	0.00	0.00	6c	1	120.02	2,544,297.98		0	3,238,331
EM	8 Dump Truck (Rock Fill Borrow Are	3,957,019	CY	0.93	C1	0.009	35,613	4,451.6	11.88	423,084.46	0.00	0.00	3e1	1	91.31	3,251,975.80		0	3,675,060

EAA Reservoir A-1 Basis of Design Report | Case: 6 | January, 2006

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, U/S Rockfill Embankment (OG +24) "Incl. 15 Foot Wave Break Bench OG +16," With 30' Cutoff Wall, and Seepage Canal 15 Feet Dee
South Side Embankment and Canal Redesign Adjustmen

With Seepage Control	12.95 Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75 Miles	=	46,200	Total Linear Feet

					Labor						Material		Equipment						
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost
EM	8 Dozer Angle Blade (New Embankr	3,957,019	CY	1.03	B2	0.014	53,420	3,338.7	16.37	874,481.39	0.00	0.00	6c	1	120.02	3,205,815.45		0	4,080,297
EM	10 Common Borrow (Top Soil - Peat)	183,931	CY	3.65	C6	0.047	8,645	308.7	14.17	122,496.50	0.00	0.00	3f	3	63.57	549,560.87		0	672,057
EM	10 W/O Common Borrow (Top Soil - Peat)	89,166	CY	3.65	C6	0.047	4,191	149.7	14.17	59,383.66	0.00	0.00	3f	3	63.57	266,415.27		0	325,799
02370	**Erosion and Sedimentation Contro**																		
	Rip Rap																		
EM	8 12' Rad. Crawler Mtd. (Prod. Blast	449,138	CY	29.89	B2	0.258	115,878	7,242.3	16.37	1,896,916.04	18.75	8,421,336.00	4b	2	26.80	3,104,998.56		0	13,423,251
EM	8 Hauling (5 Miles)	449,138	CY	2.47	C1	0.040	17,966	2,245.7	11.88	213,430.34	0.00	0.00	3e	1	49.87	895,998.63		0	1,109,429
02700	**Bases, Ballasts, Pavements and Appurtenance:**																		
EM	8 w/o Crusher (Transition Borrow Area	65775	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		822,189	822,189
EM	8 w/o Aggregate Base 1-1/2" Stone, 12"	197325	SY	1.13	B7a	0.019	3,774	52.4	16.28	61,434.04	0.00	0.00	7a+	7	42.93	162,009.17		0	223,443
02910	**Plant Preparation**																		
EM	10 Fine Grading	623,589	SY	0.32	B5a	0.008	4,989	311.8	17.30	86,279.79	0.00	0.00	7d	1	22.97	114,592.18		0	200,872
EM	10 W/O Fine Grading	235,158	SY	0.32	B5a	0.008	1,881	117.6	17.30	32,536.46	0.00	0.00	7d	1	22.97	43,213.18		0	75,750
02920	**Lawns and Grasses**																		
EM	10 Hydro or Air Seeding w/ Mulch anc	623,589	SY	0.24	C4a	0.003	1,871	3.0	15.49	28,978.19	0.15	93,538.37	3j5	2	14.11	26,393.08		0	148,910
EM	10 W/O Hydro or Air Seeding w/ Mulch anc	235,158	SY	0.24	C4a	0.003	705	3.0	15.49	10,927.79	0.15	35,273.70	3j5	2	14.11	9,952.94		0	56,154
	Subtotal Site Constructior						1,934,905			$30,459,582		$24,456,411				$118,253,973	$12,444,124	$8,700,726	$194,314,818
3	**Concrete**																		
03050	**Basic Concrete Materials and Methods**																		
03100	**Concrete and Forms and Accessories**																		
03200	**Concrete Reinforcement**																		
03300	**Cast-In-Place Concrete**																		
03310	**Structural Concrete**																		
	Roller Compacted Concrete																		
EM	11 Mass Placement, 1' Lift, 12" Layer	56,752	CY	1.07	B5	0.009	511	21.3	16.91	8,635.40		0.00	6f1	2	102.16	52,179.16		0	60,815
EM	11 Sloped Face, Nonformed, 1' Lift	216,068	CY	5.00	B5	0.042	9,075	378.1	16.91	153,425.68		0.00	6f1	2	102.16	927,070.77		0	1,080,496
EM	11 Roller Compacted Concrete, 1.5"-:	272,820	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	12,276,910.80				0.00		0	12,276,911
EM	11 Dump Truck (18 CY) Conveying M	272,820	CY	1.24	C1	0.012	3,274	409.2	11.88	38,893.25		0.00	3e1	1	91.31	298,947.21		0	337,840
EM	11 Truck Mtd. Hydraulic Crane 100 Tc	56,752	CY	0.59	D18a	0.025	1,419	35.5	17.43	24,735.39		0.00	8g	4	6.32	8,961.06		0	33,696
EM	11 Surface Prep. Vacuum Truck	*85,128*	SY	0.18	C6a-	0.006	511	25.5	17.72	9,049.80		0.00	3j2	1	12.86	6,566.61		0	15,616
EM	11 Surface Prep. Water Clean	85,128	SY	0.22	C1a	0.008	681	28.4	15.63	10,646.69		0.00	3j3	4	11.43	7,785.08		0	18,432
EM	11 Surface Prep. Water Blast	85,128	SY	0.87	C1a	0.030	2,554	106.4	15.63	39,925.09		0.00	3j4	5	13.25	33,835.36		0	73,760
EM	**11 Concrete Batch Plant and Delive**	**281,005**	**CY**	**26.15**	**C6a**	**0.480**	**134,882**	**2,107.5**	**14.44**	**1,947,026.39**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**5,400,396.87**		**0**	**7,347,423**
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	38,346	CY	1.07	B5	0.009	345	14.4	16.91	5,834.73		0.00	6f1	2	102.16	35,256.19		0	41,091
EM	11 W/O Sloped Face, Nonformed, 1' Lift	145,992	CY	5.00	B5	0.042	6,132	255.5	16.91	103,666.00		0.00	6f1	2	102.16	626,399.17		0	730,065
EM	11 W/O Roller Compacted Concrete, 1.5"-:	184,338	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	8,295,210.00				0.00		0	8,295,210
EM	11 W/O Dump Truck (18 CY) Conveying M	184,338	CY	1.24	C1	0.012	2,212	276.5	11.88	26,279.23		0.00	3e1	1	91.31	201,991.36		0	228,271
EM	11 W/O Truck Mtd. Hydraulic Crane 100 Tc	38,346	CY	0.44	D18a	0.025	959	24.0	17.43	16,713.10		0.00	8g	0	0.00	0.00		0	16,713
EM	11 W/O Surface Prep. Vacuum Truck	*57,519*	SY	0.11	C6a-	0.006	345	17.3	17.72	6,114.73		0.00	3j2	0	0.00	0.00		0	6,115
EM	11 W/O Surface Prep. Water Clean	57,519	SY	0.22	C1a	0.008	460	19.2	15.63	7,193.71		0.00	3j3	4	11.43	5,260.19		0	12,454
EM	11 W/O Surface Prep. Water Blast	57,519	SY	0.87	C1a	0.030	1,726	71.9	15.63	26,976.41		0.00	3j4	5	13.25	22,861.73		0	49,838
EM	**11 W/O Concrete Batch Plant and Delive**	**189,868**	**CY**	**26.15**	**C6a**	**0.480**	**91,137**	**1,424.0**	**14.44**	**1,315,558.37**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,648,916.81**		**0**	**4,964,475**
03400	**Precast Concrete**																		
03500	**Cementitious Decks and Underlay**																		
03600	**Grouts**																		
03900	**Concrete Restorations and Cleaning**																		
		0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0
	Subtotal Concrete						256,221			$3,740,674		$20,572,121				$11,276,428	$0	$0	$35,589,222
	Construction Subtotal (Direct Costs						2,191,126			$34,200,256		$45,028,532				$129,530,401	$12,444,124	$42,036,812	**$263,240,126**

Indirect Costs			
Sales Tax	6%	of purchased materials + Rental Equipment	10,473,536
Overhead and Profit	17%	of construction cost + general requirements	43,692,259
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	11,109,207
Project Reserve	5%	of construction cost	16,425,756
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	103,482,265
Construction Subtotal Indirects			**$185,183,024**
Total Construction (Directs and Indirect Costs			**$448,423,150**
Permits			0
Design	0%	of construction cost	0
Construction Managemen	0%	of construction cost	0
Total			**$448,423,150**

TABLE 8.13-7

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 7

Case: 7

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02

Project No.: 141731.0910

Opinion Of Probable Cost

Revision No.: 2 eservoir A-1, U/S Rockfill Embankment (OG +22) "Incl. 15 Foot Wave Break Bench OG +16," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

Date: 07/27/05 South Side Embankment and Canal Redesign Adjustment

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				57,763	985,920	0	6,095,582	0	0	7,081,502	0.9467	6,704,384	13,785,886
	Subtotal				57,763	$985,920	$0	$6,095,582	$0	$0	$7,081,502		$6,704,384	$13,785,886
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				184,168	3,024,397	2,297,434	13,031,974	0	0	18,353,805	0.9467	17,376,392	35,730,197
	Subtotal				184,168	$3,024,397	$2,297,434	$13,031,974	$0	$0	$18,353,805		$17,376,392	$35,730,197
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.9467	3,415,960	7,024,067
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$3,415,960	$7,024,067
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				652,397	10,232,664	6,237,828	41,732,323	0	0	58,202,814	0.9467	55,103,285	113,306,099
	Subtotal				652,397	$10,232,664	$6,237,828	$41,732,323	$0	$0	$58,202,814		$55,103,285	$113,306,099
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				57,044	918,526	0	1,627,925	0	7,276,195	9,822,647	0.9467	9,299,552	19,122,199
	Subtotal				57,044	$918,526	$0	$1,627,925	$0	$7,276,195	$9,822,647		$9,299,552	$19,122,199
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				80,250	1,226,189	5,925,920	2,115,591	11,275,202	0	20,542,902	0.9467	19,448,911	39,991,813
	Subtotal				80,250	$1,226,189	$5,925,920	$2,115,591	$11,275,202	$0	$20,542,902		$19,448,911	$39,991,813
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				32,427	516,882	0	1,184,628	1,125,927	0	2,827,438	0.9467	2,676,866	5,504,304
	Subtotal				32,427	$516,882	$0	$1,184,628	$1,125,927	$0	$2,827,438		$2,676,866	$5,504,304
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				590,654	9,211,525	6,917,526	35,424,529	0	822,189	52,375,769	0.9467	49,586,553	101,962,322
	Subtotal				590,654	$9,211,525	$6,917,526	$35,424,529	$0	$822,189	$52,375,769		$49,586,553	$101,962,322
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				19,690	300,395	110,794	902,190	0	0	1,313,378	0.9467	1,243,436	2,556,814
	Subtotal				19,690	$300,395	$110,794	$902,190	$0	$0	$1,313,378		$1,243,436	$2,556,814
11	**Embankment Construction (Cutoff Wall Cap, Concrete Face, and Parapet)**													
	Materials and Methods				256,221	3,740,674	20,572,121	11,276,428	0	0	35,589,222	0.9467	33,693,956	69,283,178
	Subtotal				256,221	$3,740,674	$20,572,121	$11,276,428	$0	$0	$35,589,222		$33,693,956	$69,283,178
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				929	11,560	0	52,728	0	103,249	167,538	0.9467	158,616	326,154
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$158,616	$326,154
	Total				**1,964,998**	**$30,651,031**	**$42,061,622**	**$116,569,706**	**$12,401,129**	**$8,201,633**	**$209,885,121**		**$198,707,910**	**$408,593,032**

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

Case: 7
Opinion Of Probable Cost

EAA Reservoir A-1, U/S Rockfill Embankment (OG +22) "Incl. 15 Foot Wave Break Bench OG +16," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control 12.95 Miles = 68,376 Total Linear Feet
Without Seepage Control (South Side) 8.75 Miles = 46,200 Total Linear Feet

						Labor						Material		Equipment							
CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1		**General Requirements**																			
		Mobilization	1	LS	2,098,851						0.00		0.00						2,098,851	2,098,851	
		Supervision	1	LS	10,494,256						0.00		0.00						10,494,256	10,494,256	
		Temporary Construction Facilities	1	LS	5,247,128						0.00		0.00						5,247,128	5,247,128	
		Temporary Utilities	1	LS	3,148,277						0.00		0.00						3,148,277	3,148,277	
		Safety	1	LS	5,247,128						0.00		0.00						5,247,128	5,247,128	
		Miscellaneous	1	LS	4,197,702						0.00		0.00						4,197,702	4,197,702	
		Subtotal Mobilization									$0		$0				$0	$0	$30,433,343	$30,433,343	
2		**Site Work**																			
02225		**Demolition**																			
02230		**Site Clearing**																			
SC	1	Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
SC	1	Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439	
EM	1	Scraper (Strip Peat Embankment A	940170	CY	1.03	B5b	0.006	5,641	282.1	17.17	96,833.75	0.00	0.00	1s1	2	154.87	873,637.97		0	970,472	
EM	1	Surface Prep. (Strip Peat Embankm	1410255	SY	0.35	B5	0.008	11,282	940.2	17.22	194,314.34		0.00	3j4+	2	26	297,351.51		0	491,666	
EM	1 W/O	Scraper (Strip Peat Embankment A	400554	CY	1.03	B5b	0.006	2,403	120.2	17.17	41,255.46	0.00	0.00	1s1	2	154.87	372,208.41		0	413,464	
EM	1 W/O	Surface Prep. (Strip Peat Embankm	600831	SY	0.35	B5	0.008	4,807	400.6	17.22	82,786.50		0.00	3j4+	2	26	126,684.89		0	209,471	
EM	1	Scraper (Strip Peat Inside Bench A	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626	
EM	1 W/O	Scraper (Strip Peat Inside Bench A	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504	
EM	1	Dozer Angle Blade (Strip Peat Insid	438831	CY	1.15	B2	0.015	6,582	411	16.37	107,755	0.00	0.00	6c	1	120.02	395,025.22		0	502,780	
EM	1 W/O	Dozer Angle Blade (Strip Peat Insid	111242	CY	1.15	B2	0.015	1,669	104	16.37	27,316	0.00	0.00	6c	1	120.02	100,137.80		0	127,453	$1.19
02240		**Dewatering**																			
		Seepage Canal																			
SC	2	Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507	
SC	2	12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925	
		Production Blast																			
SC	4 w/o	Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356	
SC	4 w/o	12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578	
		Cutoff Wall																			
SC	2 w/o	Pump,12" Suction (Make-up)	31508	LF	68.94	B4	0.040	1,260	157.5	16.87	21,258.71	0.00	0.00			68.27	2,151,064.86		0	2,172,324	
SC	2 w/o	12" Dia Pipe 2,500 GPM	54722	LF	5.48	B4	0.098	5,363	670.3	16.87	90,456.10	0.00	0.00			3.83	209,584.82		0	300,041	
02300		**Earthwork**																			
02305		**Equipment Mobilization Or Demobilization**																			
	12	Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049	Other: Driving Cost for 140Miles
	12	Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417	Other: Driving Cost for 140Miles
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12	Crawler Type Drill, 4'	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736	Other: Driving Cost for 140Miles
	12	Air Compressor, 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	Excavator, Diesel Hydraulic, Crawle	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	**Concrete Batch Plant, Portable, 2**	**8**	**EA**	**3,466**	**C1**	**16.000**	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Wheeled Skid Steer, Diesel, w/ Bro	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310		**Grading**																			
02315		**Excavation and Fill**			13																
SC	2	Drilling and Blasting (Seepage Cana	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2	Excavating Cap Rock (Seepage Ca	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2	Dump Truck (Canal/Stock Pile Area	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2	Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2	Dozer Angle Blade (Stock Pile Area	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
					13.32																***18,353,805***
SC	3	Excavated - Silty, Sand, Shells (Se	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3	Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812	
SC	3	Haul to Dewater and Work Stock Pi	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,595.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719	(Avg. 1,000 FT. Around Trip)
SC	3	Dozer Angle Blade - Work Stock Pil	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790	***3,608,106***
			2,632,984																		
EM	4	Drilling and Blasting (Prod. Blast Ar	2,632,984	CY	6.02	B6	0.080	210,639	8,776.6	17.11	3,604,730.43	1.89	4,976,339.49	6h	4	34.56	7,280,617.30		0	15,861,687	
EM	4	Excavating Cap Rock (Prod. Blast A	2,632,984	CY	1.75	B2	0.015	38,935	2,433.5	16.37	637,370.02	0.00	0.00	1b	1	102.03	3,972,684.67		0	4,610,055	
EM	4	Dump Truck (Prod. Blast Stock Pile	3,291,230	CY	1.71	C1	0.017	54,540	6,817.5	11.88	647,939.71	0.00	0.00	3e1	1	91.31	4,980,292.24		0	5,628,232	(Avg. 2 Miles Around Trip)
EM	4	Hoe Ram (Stock Pile Area)	345,579	CY	1.74	A10	0.077	26,610	2,661.0	17.89	476,045.62	0.00	0.00	6i	1	5.91	125,852.73		0	601,898	
EM	4	Dozer Angle Blade (Prod. Blast Are	3,291,230	CY	1.20	B2	0.016	51,837	3,239.8	16.37	848,569.56	0.00	0.00	6c	1	120.02	3,110,823.64		0	3,959,393	30,661,265
			3,661,231																		
EM	4	Excavated - Silty, Sand, Shells (Pro	3,661,231	CY	0.80	B3	0.005	18,306	762.8	16.75	306,628.11	0.00	0.00	1n	1	144.10	2,637,974.67		0	2,944,603	
EM	4	Haul to Dewater and Work Stock Pi	4,027,354	CY	1.71	C1	0.017	66,739	8,342.4	11.88	792,859.49	0.00	0.00	3e1	1	91.31	6,094,196.56		0	6,887,056	(Avg. 2 Miles Around Trip)
EM	4	Dozer Angle Blade (Work Stock Pile	4,027,354	CY	0.69	B2	0.009	36,246	2,265.4	16.37	593,350.11	0.00	0.00	6c	1	120.02	2,175,198.88		0	2,768,549	12,600,208
			667,454																		***45,541,406***
EM	4 W/O	Drilling and Blasting (Prod. Blast Ar	667,454	CY	6.02	B6	0.080	53,396	2,224.8	17.11	913,789.15	1.89	1,261,488.23	6h	4	34.56	1,845,616.25		0	4,020,894	
EM	4 W/O	Excavating Cap Rock (Prod. Blast A	667,454	CY	1.75	B2	0.015	9,870	616.9	16.37	161,571.53	0.00	0.00	1b	1	102.03	1,007,064.52		0	1,168,636	
EM	4 W/O	Dozer Angle Blade (Prod. Blast Are	800,945	CY	1.20	B2	0.016	12,615	788.4	16.37	206,505.62	0.00	0.00	6c	1	120.02	757,041.74		0	963,547	6,153,077
			1,891,120																		***50,392,351***

Case: 7

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, U/S Rockfill Embankment (OG +22) "Incl. 15 Foot Wave Break Bench OG +16," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95	Miles	=	68,376 Total Linear Feet
Without Seepage Control (South Side)	8.75	Miles	=	46,200 Total Linear Feet

					Labor						Material		Equipment							
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	4 W/O Excavated - Silty, Sand, Shells (Pro	1,891,120	CY	0.80	B3	0.005	9,456	394.0	16.75	158,381.29	0.00	0.00	1n	1	144.10	1,362,581.66		0	1,520,963	
EM	4 W/O Haul to Dewater and Work Stock Pi	2,080,232	CY	1.71	C1	0.017	34,472	4,309.1	11.88	409,532.29	0.00	0.00	3e1	1	91.31	3,147,809.02		0	3,557,341	(Avg. 2 Miles Around Trip)
EM	4 W/O Dozer Angle Blade (Work Stock Pile	2,080,232	CY	0.69	B2	0.009	18,722	1,170.1	16.37	306,480.57	0.00	0.00	6c	1	120.02	1,123,546.08		0	1,430,027	6,508,331
EM	5 Crusher (Level Coarse Borrow Area	152,386	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,765,807	2,765,807	
EM	5 Dozer Loader (Level Coarse Borrow	152,386	CY	0.82	B5	0.013	1,943	161.9	17.22	33,463.60	0.00	0.00	6j	1	47.46	92,204.49		0	125,668	
EM	5 Dump Truck (Level Coarse Borrow	213,341	CY	1.71	C1	0.017	3,535	441.9	11.88	42,000.04	0.00	0.00	3e1	1	91.31	322,827.08		0	364,827	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankm	213,341	CY	1.03	B2	0.014	2,880	180.0	16.37	47,147.19	0.00	0.00	6c	1	120.02	172,839.79		0	219,987	3,476,290
EM	5 Crusher (Bedding Borrow Area)	139,783	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,537,056	2,537,056	
EM	5 Dozer Loader (Bedding Borrow Are	139,783	CY	0.82	B5	0.013	1,782	148.5	17.22	30,695.94	0.00	0.00	6j	1	47.46	84,578.56		0	115,274	
EM	5 Dump Truck (Bedding Borrow Area)	195,696	CY	1.71	C1	0.017	3,243	405.4	11.88	38,526.35	0.00	0.00	3e1	1	91.31	296,127.09		0	334,653	(Avg. 2 Miles Around Trip)
EM	5 Dozer Angle Blade (New Embankm	195,696	CY	1.03	B2	0.014	2,642	165.1	16.37	43,247.80	0.00	0.00	6c	1	120.02	158,544.77		0	201,793	3,188,777
EM	5 Crusher (Granular Toe Borrow Area	106,667	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,973,331	1,973,331	
EM	5 Dozer Loader (Granular Toe Borrow	106,667	CY	0.82	B5	0.013	1,360	113.3	17.22	23,423.71	0.00	0.00	6j	1	47.46	64,540.91		0	87,965	
EM	5 Dump Truck (Granular Toe Borrow	149,333	CY	1.71	C1	0.017	2,475	309.3	11.88	29,399.01	0.00	0.00	3e1	1	91.31	225,971.12		0	255,370	(Avg. 2 Miles Around Trip)
EM	5 Backhoe Loader (New Embankmen	149,333	CY	3.84	B4-	0.160	23,893	995.6	16.65	397,903.25	0.00	0.00	2a	1	7.33	175,172.35		0	573,076	
EM	5 Compacting (New Embankment Are	149,333	CY	1.79	A1	0.089	13,291	1,661.3	17.51	232,719.34	0.00	0.00	8b2	1	2.64	35,119.16		0	267,839	3,157,580 3,157,580
EM	6 Clean Cap Rock Surface (Embankn	432,820	SY	0.87	C1a	0.030	12,985	541.0	15.63	202,992.62		0.00	3j4	5	13.25	172,030.41		0	375,023	
EM	6 Cement Grout Cap Rock (Embankn	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 W/O Clean Cap Rock Surface (Embankn	323,400	SY	0.87	C1a	0.030	9,702	404.3	15.63	151,674.60		0.00	3j4	5	13.25	128,539.86		0	280,214	
EM	6 W/O Cement Grout Cap Rock (Embankn	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0	
EM	6 w/o Cutoff Wall (Embankment Area)	68,376	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	5,470,080	0	5,470,080	(Quoted for 2' Wide at $40/LF)
EM	6 w/o Lean Concrete Fill In Cap Rock Cut	91,168	CY	67.54	D8	0.137	12,490	260.2	17.69	220,885.93	65.00	5,925,920.00	8b1	2	0.83	10,376.69		0	6,157,183	
EM	6 w/o **Concrete Batch Plant and Deliver**	93,903	CY	26.15	C6a	0.480	45,073	704.3	14.44	650,635.38	0.00	0.00	8h1	6	40.04	1,804,643.90		0	2,455,279	
EM	6 w/o Cutoff Wall (Embankment Area)	2,051,280	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	5,128,200	0	5,128,200	(Soil Bentonite)
SC	6 w/o Cutoff Wall (Through Limestone)	15,043	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	676,922	0	676,922	20,542,902
EM	7 Dozer Loader (Sand Horiz. Filter)	185,299	CY	0.46	B2	0.006	1,112	69.5	16.37	18,200.06	0.00	0.00	6c	1	120.02	66,720.74		0	84,921	
EM	7 Dump Truck (Sand Horiz. Filter)	185,299	CY	0.93	C1	0.009	1,668	208.5	11.88	19,812.16	0.00	0.00	3e1	1	91.31	152,283.26		0	172,095	
EM	7 Wash and Screen (Sand Horiz. Filte	185,299	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	728,225	0	728,225	
EM	7 Dozer Angle Blade (Sand Horiz. Filt	185,299	CY	0.69	B2	0.009	1,668	104.2	16.37	27,300.10	0.00	0.00	6c	1	120.02	100,081.11		0	127,381	
EM	7 Dozer and Dump Trucks (Sand Hor	185,299	CY	1.75	C6	0.024	4,355	155.5	14.17	61,703.63	0.00	0.00	3f1	2	60.38	262,935.52		0	324,639	
EM	7 Dozer Angle Blade (Sand Horiz. Filt	185,299	CY	0.69	B2	0.009	1,668	104.2	16.37	27,300.10	0.00	0.00	6c	1	120.02	100,081.11		0	127,381	
EM	7 Compact (Sand Horiz. Filter)	185,299	CY	0.79	B5	0.009	1,668	139.0	17.22	28,723.19	0.00	0.00	6g1	1	70.54	117,641.66		0	146,365	
EM	7 Dozer Loader (Sand Vert. Filter)	101,196	CY	0.46	B2	0.006	607	37.9	16.37	9,939.52	0.00	0.00	6c	1	120.02	36,437.89		0	46,377	
EM	7 Dump Truck (Sand Vert. Filter)	101,196	CY	0.93	C1	0.009	911	113.8	11.88	10,819.93	0.00	0.00	3e1	1	91.31	83,165.76		0	93,986	
EM	7 Wash and Screen (Sand Vert. Filter	101,196	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	397,702	0	397,702	
EM	7 Dozer Angle Blade (Sand Vert. Filte	101,196	CY	0.69	B2	0.009	911	56.9	16.37	14,909.28	0.00	0.00	6c	1	120.02	54,656.84		0	69,566	
EM	7 Dozer and Dump Trucks (Sand Vert	101,196	CY	1.75	C6	0.024	2,378	84.9	14.17	33,697.92	0.00	0.00	3f1	2	60.38	143,595.78		0	177,294	
EM	7 Backhoe (Sand Vert. Filter)	101,196	CY	3.28	B4a	0.153	15,483	322.6	17.08	264,476.49	0.00	0.00	2	1	4.33	67,028.75		0	331,505	
EM	8 Dozer and Dump Trucks (Max. 6" R	1,843,417	CY	1.75	C6	0.024	43,320	1,547.2	14.17	613,848.63	0.00	0.00	3f1	2	60.38	2,615,771.77		0	3,229,620	
EM	8 Dozer With Ripper Attach, (Max. 6"	1,843,417	CY	2.04	B5	0.020	36,868	3,072.4	17.22	634,995.70	0.00	0.00	6g	2	84.81	3,126,873.96		0	3,761,870	
EM	8 Dozer Angle Blade (Max. 6" Randor	1,843,417	CY	0.69	B2	0.009	16,591	1,036.9	16.37	271,590.62	0.00	0.00	6c	1	120.02	995,640.86		0	1,267,231	
EM	8 Compact (Max. 6" Random Fill)	1,843,417	CY	0.79	B5	0.009	16,591	1,382.6	17.22	285,748.06	0.00	0.00	6g1	1	70.54	1,170,339.18		0	1,456,087	
EM	8 W/O Dozer and Dump Trucks (Max. 6" R	1,591,590	CY	1.75	C6	0.024	37,402	1,335.8	14.17	529,991.51	0.00	0.00	3f1	2	60.38	2,258,434.36		0	2,788,426	
EM	8 W/O Dozer With Ripper Attach, (Max. 6"	1,591,590	CY	2.04	B5	0.020	31,832	2,652.7	17.22	548,249.70	0.00	0.00	6g	2	84.81	2,699,715.49		0	3,247,965	
EM	8 W/O Dozer Angle Blade (Max. 6" Randor	1,591,590	CY	0.69	B2	0.009	14,324	895.3	16.37	234,488.95	0.00	0.00	6c	1	120.02	859,627.57		0	1,094,117	
EM	8 W/O Compact (Max. 6" Random Fill)	1,591,590	CY	0.79	B5	0.009	14,324	1,193.7	17.22	246,712.37	0.00	0.00	6g1	1	70.54	1,010,460.56		0	1,257,173	
EM	8 Dozer and Dump Trucks (Mass Rar	2,495,040	CY	1.75	C6	0.024	58,633	2,094.1	14.17	830,835.92	0.00	0.00	3f1	2	60.38	3,540,412.17		0	4,371,248	
EM	8 Dozer Angle Blade (Mass Random	2,495,040	CY	0.69	B2	0.009	22,455	1,403.5	16.37	367,594.28	0.00	0.00	6c	1	120.02	1,347,586.61		0	1,715,181	
EM	8 Compact (Mass Random Fill)	2,495,040	CY	0.79	B5	0.009	22,455	1,871.3	17.22	386,756.19	0.00	0.00	6g1	1	70.54	1,584,038.46		0	1,970,795	26,159,713
EM	8 W/O Dozer and Dump Trucks (Mass Rar	1,361,976	CY	1.75	C6	0.024	32,006	1,143.1	14.17	453,531.20	0.00	0.00	3f1	2	60.38	1,932,616.69		0	2,386,148	
EM	8 W/O Dozer Angle Blade (Mass Random	1,361,976	CY	0.69	B2	0.009	12,258	766.1	16.37	200,659.92	0.00	0.00	6c	1	120.02	735,611.63		0	936,272	
EM	8 W/O Compact (Mass Random Fill)	1,361,976	CY	0.79	B5	0.009	12,258	1,021.5	17.22	211,119.90	0.00	0.00	6g1	1	70.54	864,684.40		0	1,075,804	30,557,937
EM	8 Dozer Loader (Rock Fill Borrow Are	2,271,358	CY	1.15	B2	0.015	34,070	2,129.4	16.37	557,732.04	0.00	0.00	6c	1	120.02	2,044,624.38		0	2,602,356	
EM	8 Dump Truck (Rock Fill Borrow Area	3,179,902	CY	0.93	C1	0.009	28,619	3,577.4	11.88	339,995.08	0.00	0.00	3e1	1	91.31	2,613,321.65		0	2,953,317	
EM	8 Dozer Angle Blade (New Embankm	3,179,902	CY	1.03	B2	0.014	42,929	2,683.0	16.37	702,742.36	0.00	0.00	6c	1	120.02	2,576,226.72		0	3,278,969	8,834,642
EM	10 Common Borrow (Top Soil - Peat)	167,521	CY	3.65	C6	0.047	7,873	281.2	14.17	111,567.44	0.00	0.00	3f	3	63.57	500,529.41		0	612,097	
EM	10 W/O Common Borrow (Top Soil - Peat)	78,540	CY	3.65	C6	0.047	3,691	131.8	14.17	52,306.85	0.00	0.00	3f	3	63.57	234,666.30		0	286,973	899,070
02370	**Erosion and Sedimentation Control**																			
	Rip Rap																			
EM	8 12' Rad. Crawler Mtd. (Prod. Blast T	368,935	CY	29.89	B2	0.258	95,185	5,949.1	16.37	1,558,181.03	18.75	6,917,526.00	4b	2	26.80	2,550,534.53		0	11,026,242	
EM	8 Hauling (5 Miles)	368,935	CY	2.47	C1	0.040	14,757	1,844.7	11.88	175,317.78	0.00	0.00	3e	1	49.87	735,998.88		0	911,317	11,937,558
02700	**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o Crusher (Transition Borrow Area)	65775	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		822,189	822,189	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" T	197325	SY	1.13	B7a	0.019	3,774	52.4	16.28	61,434.04	0.00	0.00	7a+	7	42.93	162,009.17		0	223,443	1,045,632
02910	**Plant Preparation**																			
EM	10 Fine Grading	503,931	SY	0.32	B5a	0.008	4,031	252.0	17.30	69,723.91	0.00	0.00	7d	1	22.97	92,603.55		0	162,327	
EM	10 W/O Fine Grading	234,696	SY	0.32	B5a	0.008	1,878	117.3	17.30	32,472.54	0.00	0.00	7d	1	22.97	43,128.28		0	75,601	

Case: 7

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order No. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, U/S Rockfill Embankment (OG +22) "Incl. 15 Foot Wave Break Bench OG +16," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep
South Side Embankment and Canal Redesign Adjustment

With Seepage Control	12.95	Miles	=	68,376	Total Linear Feet
Without Seepage Control (South Side)	8.75	Miles	=	46,200	Total Linear Feet

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Material: Unit Cost	Material Cost	Equipment: Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
02920	**Lawns and Grasses**																			
EM	10 Hydro or Air Seeding w/ Mulch and	503,931	SY	0.24	C4a	0.003	1,512	3.0	15.49	23,417.68	0.15	75,589.67	3j5	2	14.11	21,328.62		0	120,336	358,264
EM	10 W/O Hydro or Air Seeding w/ Mulch and	234,696	SY	0.24	C4a	0.003	704	3.0	15.49	10,906.32	0.15	35,204.40	3j5	2	14.11	9,933.38		0	56,044	414,308
	Subtotal Site Construction						1,708,777			$26,910,357		$21,489,501				$105,293,278	$12,401,129	$8,201,633	$174,295,899	
3	**Concrete**																			
03050	**Basic Concrete Materials and Methods**																			
03100	**Concrete and Forms and Accessories**																			
03200	**Concrete Reinforcement**																			
03300	**Cast-In-Place Concrete**																			
03310	**Structural Concrete**																			
	Roller Compacted Concrete																			
EM	11 Mass Placement, 1' Lift, 12" Layer	56,752	CY	1.07	B5	0.009	511	21.3	16.91	8,635.40		0.00	6f1	2	102.16	52,179.16		0	60,815	
EM	11 Sloped Face, Nonformed, 1' Lift	216,068	CY	5.00	B5	0.042	9,075	378.1	16.91	153,425.68		0.00	6f1	2	102.16	927,070.77		0	1,080,496	
EM	11 Roller Compacted Concrete, 1.5"-2'	272,820	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	12,276,910.80				0.00		0	12,276,911	100 lbs of cement by volume
EM	11 Dump Truck (18 CY) Conveying Ma	272,820	CY	1.24	C1	0.012	3,274	409.2	11.88	38,893.25		0.00	3e1	1	91.31	298,947.21		0	337,840	(15 Min. Cycles)
EM	11 Truck Mtd. Hydraulic Crane 100 Ton	56,752	CY	0.59	D18a	0.025	1,419	35.5	17.43	24,735.39		0.00	8g	4	6.32	8,961.06		0	33,696	$50.55
EM	11 Surface Prep. Vacuum Truck	*85,128*	SY	0.18	C6a-	0.006	511	25.5	17.72	9,049.80		0.00	3j2	1	12.86	6,566.61		0	15,616	
EM	11 Surface Prep. Water Clean	85,128	SY	0.22	C1a	0.008	681	28.4	15.63	10,646.69		0.00	3j3	4	11.43	7,785.08		0	18,432	
EM	11 Surface Prep. Water Blast	85,128	SY	0.87	C1a	0.030	2,554	106.4	15.63	39,925.09		0.00	3j4	5	13.25	33,835.36		0	73,760	*$50.94*
EM	**11 Concrete Batch Plant and Deliver**	**281,005**	**CY**	**26.15**	**C6a**	**0.480**	**134,882**	**2,107.5**	**14.44**	**1,947,026.39**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**5,400,396.87**		**0**	**7,347,423**	$77.87
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	38,346	CY	1.07	B5	0.009	345	14.4	16.91	5,834.73		0.00	6f1	2	102.16	35,256.19		0	41,091	
EM	11 W/O Sloped Face, Nonformed, 1' Lift	145,992	CY	5.00	B5	0.042	6,132	255.5	16.91	103,666.00		0.00	6f1	2	102.16	626,399.17		0	730,065	
EM	11 W/O Roller Compacted Concrete, 1.5"-2'	184,338	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	8,295,210.00				0.00		0	8,295,210	100 lbs of cement by volume
EM	11 W/O Dump Truck (18 CY) Conveying Ma	184,338	CY	1.24	C1	0.012	2,212	276.5	11.88	26,279.23		0.00	3e1	1	91.31	201,991.36		0	228,271	(15 Min. Cycles)
EM	11 W/O Truck Mtd. Hydraulic Crane 100 Ton	38,346	CY	0.44	D18a	0.025	959	24.0	17.43	16,713.10		0.00	8g	0	0.00	0.00		0	16,713	
EM	11 W/O Surface Prep. Vacuum Truck	*57,519*	SY	0.11	C6a-	0.006	345	17.3	17.72	6,114.73		0.00	3j2	0	0.00	0.00		0	6,115	
EM	11 W/O Surface Prep. Water Clean	57,519	SY	0.22	C1a	0.008	460	19.2	15.63	7,193.71		0.00	3j3	4	11.43	5,260.19		0	12,454	
EM	11 W/O Surface Prep. Water Blast	57,519	SY	0.87	C1a	0.030	1,726	71.9	15.63	26,976.41		0.00	3j4	5	13.25	22,861.73		0	49,838	$50.88
EM	**11 W/O Concrete Batch Plant and Deliver**	**189,868**	**CY**	**26.15**	**C6a**	**0.480**	**91,137**	**1,424.0**	**14.44**	**1,315,558.37**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,648,916.81**		**0**	**4,964,475**	$50.51 **$77.81**
03400	**Precast Concrete**																			
03500	**Cementitious Decks and Underlay**																			
03600	**Grouts**																			
03900	**Concrete Restorations and Cleaning**																			
		0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0	***35,589,222***
	Subtotal Concrete						256,221			$3,740,674		$20,572,121				$11,276,428	$0	$0	$35,589,222	
	Construction Subtotal (Direct Costs)						1,964,998			$30,651,031		$42,061,622				$116,569,706	$12,401,129	$38,634,976	**$240,318,464**	

Indirect Costs			
Sales Tax	6%	of purchased materials + Rental Equipment	9,517,880
Overhead and Profit	16%	of construction cost + general requirements	39,376,752
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	10,122,458
Project Reserve	5%	of construction cost	14,966,778
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	94,290,700
Construction Subtotal Indirects			**$168,274,568**
Total Construction (Directs and Indirect Costs)			**$408,593,032**
Permits			0
Design	0%	of construction cost	0
Construction Management	0%	of construction cost	0
Total			**$408,593,032**

TABLE 8.13-8

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 8

Case: 8

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No.: 141731.0910
Revision No.: 2
Date: 07/27/05

Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG+23) "Including 15 Foot Wave Break Bench OG +16", With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	Strip Peat													
	Materials and Methods				69,886	1,194,783	0	7,864,067	0	0	9,058,850	0.9472	8,580,549	17,639,398
	Subtotal				69,886	$1,194,783	$0	$7,864,067	$0	$0	$9,058,850		$8,580,549	$17,639,398
2	Seepage Collection Canal Construction (Cap Rock Removal)													
	Materials and Methods				186,319	3,060,682	2,297,434	11,821,449	0	0	17,179,565	0.9472	16,272,496	33,452,061
	Subtotal				186,319	$3,060,682	$2,297,434	$11,821,449	$0	$0	$17,179,565		$16,272,496	$33,452,061
3	Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)													
	Materials and Methods				37,359	541,910	0	3,562,699	0	0	4,104,608	0.9472	3,887,888	7,992,496
	Subtotal				37,359	$541,910	$0	$3,562,699	$0	$0	$4,104,608		$3,887,888	$7,992,496
4	Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)													
	Materials and Methods				441,548	7,176,534	5,596,335	23,407,654	0	0	36,180,524	0.9472	34,270,218	70,450,742
	Subtotal				441,548	$7,176,534	$5,596,335	$23,407,654	$0	$0	$36,180,524		$34,270,218	$70,450,742
5	Embankment Construction (Cap Rock Crushing)													
	Materials and Methods				28,166	435,178	0	1,847,460	0	5,038,847	7,321,484	0.9472	6,934,915	14,256,399
	Subtotal				28,166	$435,178	$0	$1,847,460	$0	$5,038,847	$7,321,484		$6,934,915	$14,256,399
6	Embankment Construction (Surface Preparation / Cutoff Wall)													
	Materials and Methods				204,014	3,202,598	12,262,624	4,590,538	19,154,784	0	39,210,544	0.9472	37,140,256	76,350,800
	Subtotal				204,014	$3,202,598	$12,262,624	$4,590,538	$19,154,784	$0	$39,210,544		$37,140,256	$76,350,800
7	Embankment Construction (Sand filters and Drains)													
	Materials and Methods				0	0	0	0	0	0	0	0.9472	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
8	Embankment Construction (Rock Fill)													
	Materials and Methods				86,473	1,315,876	0	5,818,834	0	806,667	7,941,377	0.9472	7,522,078	15,463,455
	Subtotal				86,473	$1,315,876	$0	$5,818,834	$0	$806,667	$7,941,377		$7,522,078	$15,463,455
9	Embankment Construction (Broken Cap Rock And Silty Sand, Shell, Etc. Soils)													
	Materials and Methods				49,267	754,241	0	1,624,953	0	0	2,379,194	0.9472	2,253,575	4,632,769
	Subtotal				49,267	$754,241	$0	$1,624,953	$0	$0	$2,379,194		$2,253,575	$4,632,769
10	Embankment Construction (Topsoil and Seeding)													
	Materials and Methods				0	0	0	0	0	0	0	0.9472	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
11	Embankment Construction (RCC And Concrete)													
	Materials and Methods				1,669,818	25,416,274	99,870,186	53,569,526	0	0	178,855,987	0.9472	169,412,519	348,268,505
	Subtotal				1,669,818	$25,416,274	$99,870,186	$53,569,526	$0	$0	$178,855,987		$169,412,519	$348,268,505
12	Equipment Mobilization Or Demobilization													
	Materials and Methods				737	9,103	0	43,887	0	87,069	140,059	0.9472	132,664	272,723
	Subtotal				737	$9,103	$0	$43,887	$0	$87,069	$140,059		$132,664	$272,723
	Total				**2,773,587**	**$43,107,179**	**$120,026,579**	**$114,151,067**	**$19,154,784**	**$5,932,582**	**$302,372,191**		**$286,407,156**	**$588,779,348**

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, RCC (OG+23) "Including 15 Foot Wave Break Bench OG +16", With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total Linear Feet

CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	Labor: M-H per Unit	Labor: Man-Hours	Labor: Duration Days	Labor: Average Wage Rate	Labor: Labor Cost	Material: Unit Cost	Material: Material Cost	Equipment: Code	Equipment: No.	Equipment: Avg. Cost ($/hr)	Equipment: Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1		**General Requirements**																			
		Mobilization	1	LS	3,023,722						0.00		0.00						3,023,722	3,023,722	
		Supervision	1	LS	15,118,610						0.00		0.00						15,118,610	15,118,610	
		Temporary Construction Facilities	1	LS	7,559,305						0.00		0.00						7,559,305	7,559,305	
		Temporary Utilities	1	LS	4,535,583						0.00		0.00						4,535,583	4,535,583	
		Safety	1	LS	7,559,305						0.00		0.00						7,559,305	7,559,305	
		Miscellaneous	1	LS	6,047,444						0.00		0.00						6,047,444	6,047,444	
		Subtotal Mobilization									$0		$0				$0	$0	$43,843,968	$43,843,968	
2		**Site Work**																			
02225		**Demolition**																			
02230		**Site Clearing**																			
SC	1	Scraper (Strip Peat Canal Area)	860,444	CY	2.06	B5b	0.012	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
SC	1	Scraper (Strip Peat Bench Area)	1720889	CY	1.55	B5b	0.009	15,488	774.4	17.17	265,867.01	0.00	0.00	1s1	2	154.87	2,398,662.81		0	2,664,530	
EM	1	Scraper (Strip Peat Embankment A	1463616	CY	1.03	B5b	0.006	8,782	439.1	17.17	150,746.59	0.00	0.00	1s1	2	154.87	1,360,041.81		0	1,510,788	
EM	1	Surface Prep. (Strip Peat Embankn	2195424	SY	0.35	B5	0.008	17,563	1,463.6	17.22	302,500.15		0.00	3j4+	2	26	462,903.97		0	765,404	
EM	1	Scraper (Strip Peat Inside Bench A	2581333	CY	0.69	B5b	0.004	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
EM	1	Dozer Angle Blade (Strip Peat Insic	493504	CY	1.15	B2	0.015	7,403	463	16.37	121,180	0.00	0.00	6c	1	120.02	444,241		0	565,421	
02240		**Dewatering**																			
		Seepage Canal																			
SC	2	Pump,12" Suction (Make-up)	39,146	LF	40.86	B4	0.040	1,566	195.7	16.87	26,411.75	0.00	0.00			40.19	1,573,116.00		0	1,599,528	
SC	2	12" Dia Pipe 2,500 GPM	55,060	LF	5.48	B4	0.098	5,396	674.5	16.87	91,014.74	0.00	0.00			3.83	210,879.19		0	301,894	
		Production Blast																			
SC	4	Pump,12" Suction (Make-up)	26136	LF	40.86	B4	0.040	1,045	130.7	16.87	17,633.95	0.00	0.00			40.19	1,050,300.00		0	1,067,934	
SC	4	12" Dia Pipe 2,500 GPM	92928	LF	5.48	B4	0.098	9,107	1,138.4	16.87	153,611.38	0.00	0.00			3.83	355,914.24		0	509,526	
		Cutoff Wall																			
SC	2 w/o	Pump,12" Suction (Make-up)	*31944*	LF	40.86	B4	0.040	1,278	159.7	16.87	21,552.61	0.00	0.00			40.19	1,283,700.00		0	1,305,253	
SC	2 w/o	12" Dia Pipe 2,500 GPM	*72,019*	LF	5.48	B4	0.098	7,058	882.2	16.87	119,048.82	0.00	0.00			3.83	275,833.54		0	394,882	
02300		**Earthwork**																			
02305		**Equipment Mobilization Or Demobilization**																			
	12	Dump Truck (26 Tons)	12	EA	346	C1	2.000	24	3.0	11.88	285.12	0.00	0.00	3e1	1	49.87	1,196.96		2,668	4,150	Other: Driving Cost for 140Miles
	12	Dozers (Above 150 HP)	28	EA	578	C1	2.667	75	9.3	11.88	887.15	0.00	0.00	3g	2	65.48	4,889.41		10,398	16,174	Other: Driving Cost for 140Miles
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12	Crawler Type Drill, 4"	16	EA	322	C2	6.000	96	12.0	15.55	1,492.80	0.00	0.00	6h	4	34.56	3,318.19		346	5,157	Other: Driving Cost for 140Miles
	12	Air Compressor+D294, 600 CFM	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	50 Ft Air Hose, 3" Dia.	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12	Excavator, Diesel Hydraulic, Crawl	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	**Concrete Batch Plant, Portable, 2**	**8**	**EA**	**3,466**	**C1**	**16.000**	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12	**Concrete Transit Mixer Truck**	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane up To 75 Ton	12	EA	481	C1	2.222	27	3.3	11.88	316.77	0.00	0.00	3g	2	65.48	1,745.83		3,713	5,775	Other: Driving Cost for 140Miles
	12	Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12	Attachment Clam Bucket, 1/2 CY	12	EA	578	C1	2.667	32	4.0	11.88	380.21	0.00	0.00	3g	2	65.48	2,095.46		4,456	6,932	Other: Driving Cost for 140Miles
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12	Wheeled Skid Steer, Diesel, w/ Bro	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12	Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310		**Grading**																			
02315		**Excavation and Fill**																			
SC	2	Drilling and Blasting (Seepage Can	**1,215,573**	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2	Excavating Cap Rock (Seepage Ca	**1,215,573**	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2	Dump Truck (Canal/Stock Pile Area	**1,519,467**	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2	Hoe Ram (Stock Pile Area)	**159,544**	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2	Dozer Angle Blade (Stock Pile Area	**1,519,467**	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008
																					17,179,565
SC	3	Excavated - Silty, Sand, Shells (Se	**958,372**	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3	Cut Through Limestone	**193,406**	CY	4.02	B3	0.025	4,835	201.5	16.75	80,988.93	0.00	0.00	1n	1	144.10	696,761.77		0	777,751	
SC	3	Haul to Dewater and Work Stock P	**1,266,956**	CY	1.33	C1	0.013	16,330	2,041.2	11.88	193,996.33	0.00	0.00	3e1	1	91.31	1,491,123.94		0	1,685,120	(Avg. 1,000 FT. Around Trip)
SC	3	Dozer Angle Blade - Work Stock Pi	**1,266,956**	CY	0.69	B2	0.009	11,403	712.7	16.37	186,660.65	0.00	0.00	6c	1	120.02	684,290.84		0	870,951	***4,104,608***
EM	4	Drilling and Blasting (Prod. Blast A	**2,961,024**	CY	6.02	B6	0.080	236,882	9,870.1	17.11	4,053,839.26	1.89	5,596,335.36	6h	4	34.56	8,187,700.25		0	17,837,875	
EM	4	Excavating Cap Rock (Prod. Blast A	**2,961,024**	CY	1.75	B2	0.015	43,786	2,736.6	16.37	716,779.15	0.00	0.00	1b	1	102.03	4,467,636.46		0	5,184,416	
EM	4	Dump Truck (Prod. Blast Stock Pile	**3,701,280**	CY	1.71	C1	0.017	61,335	7,666.9	11.88	728,665.71	0.00	0.00	3e1	1	91.31	5,600,780.58		0	6,329,446	(Avg. 2 Miles Around Trip)
EM	4	Hoe Ram (Stock Pile Area)	**388,634**	CY	1.74	A10	0.077	29,925	2,992.5	17.89	535,355.55	0.00	0.00	6i	1	5.91	141,532.56		0	676,888	
EM	4	Dozer Angle Blade (Prod. Blast Are	**3,701,280**	CY	1.20	B2	0.016	58,295	3,643.4	16.37	954,291.77	0.00	0.00	6c	1	120.02	3,498,397.24		0	4,452,689	34,481,314
EM	4	Excavated - Silty, Sand, Shells (Pro	**35,377**	CY	0.80	B3	0.005	177	7.4	16.75	2,962.81	0.00	0.00	1n	1	144.10	25,489.58		0	28,452	
EM	4	Haul to Dewater and Work Stock P	**38,915**	CY	1.71	C1	0.017	645	80.6	11.88	7,661.05	0.00	0.00	3e1	1	91.31	58,885.53		0	66,547	(Avg. 2 Miles Around Trip)
EM	4	Dozer Angle Blade (Work Stock Pil	**38,915**	CY	0.69	B2	0.009	350	21.9	16.37	5,733.28	0.00	0.00	6c	1	120.02	21,017.99		0	26,751	121,750

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

Case: 8
Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG+23) "Including 15 Foot Wave Break Bench OG +16", With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total Linear Feet

						Labor						Material		Equipment							
CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
																					36,180,524
EM	5	Crusher (Level Coarse Borrow Are	127,776	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,319,134	2,319,134	
EM	5	Dozer Loader (Level Coarse Borro	127,776	CY	0.82	B5	0.013	1,629	135.8	17.22	28,059.29	0.00	0.00	6j	1	47.46	77,313.63		0	105,373	
EM	5	Dump Truck (Level Coarse Borrow	178,886	CY	1.71	C1	0.017	2,964	370.6	11.88	35,217.11	0.00	0.00	3e1	1	91.31	270,691.08		0	305,908	(Avg. 2 Miles Around Trip)
EM	5	Dozer Angle Blade (New Embankm	178,886	CY	1.03	B2	0.014	2,415	150.9	16.37	39,533.00	0.00	0.00	6c	1	120.02	144,926.47		0	184,459	2,914,875
EM	5	Cruncher (Transition Borrow Area)	51110	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		927,654	927,654	
EM	5	Dozer Loader (Transition Borrow A	51110	CY	0.82	B5	0.013	652	54.3	17.22	11,223.72	0.00	0.00	6j	1	47.46	30,925.45		0	42,149	
EM	5	Dump Truck (Transition Borrow Are	71555	CY	1.71	C1	0.017	1,186	148.2	11.88	14,086.84	0.00	0.00	3e1	1	91.31	108,276.43		0	122,363	(Avg. 2 Miles Around Trip)
EM	5	Dozer Angle Blade (New Embankm	71555	CY	1.03	B2	0.014	966	60.4	16.37	15,813.20	0.00	0.00	6c	1	120.02	57,970.59		0	73,784	1,165,950
EM	5	Cruncher (Transition Borrow Area)	98736	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		1,792,058	1,792,058	
EM	5	Dozer Loader (Transition Borrow A	98736	CY	0.82	B5	0.013	1,259	104.9	17.22	21,682.18	0.00	0.00	6j	1	47.46	59,742.35		0	81,425	
EM	5	Dump Truck (Transition Borrow Are	138230	CY	1.71	C1	0.017	2,291	286.3	11.88	27,213.22	0.00	0.00	3e1	1	91.31	209,170.38		0	236,384	(Avg. 2 Miles Around Trip)
EM	5	Dozer Angle Blade (Sloped New Er	138230	CY	8.18	B2	0.107	14,804	925.3	16.37	242,349.27	0.00	0.00	6c	1	120.02	888,443.18		0	1,130,792	3,240,659
EM	6	Clean Cap Rock Surface (Embankı	297370	SY	0.87	C1a	0.030	8,921	371.7	15.63	139,466.34		0.00	3j4	5	13.25	118,193.72		0	257,660	
EM	6	Cement Grout Cap Rock (Embankr	48787	CY	78.98	B4	1.500	73,181	2,286.9	16.87	1,234,377.14	45.00	2,195,424.00	3i	2	5.78	423,183.35		0	3,852,984	
EM	**6**	**Concrete Batch Plant and Delive**	**50251**	**CY**	**26.15**	**C6a**	**0.480**	**24,120**	**376.9**	**14.44**	**348,177.85**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**965,728.36**		**0**	**1,313,906**	
EM	6 w/o	Cutoff Wall (Embankment Area)	116,160	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	9,292,800	0	9,292,800	(Quoted for 2' Wide at $40/LF)
EM	6 w/o	Lean Concrete Fill In Cap Rock Cu	154,880	CY	67.54	D8	0.137	21,219	442.1	17.69	375,250.23	65.00	10,067,200.00	8b1	2	0.83	17,628.35		0	10,460,079	
EM	**6 w/o**	**Concrete Batch Plant and Delive**	**159,526**	**CY**	**26.15**	**C6a**	**0.480**	**76,573**	**1,196.4**	**14.44**	**1,105,326.52**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,065,804.31**		**0**	**4,171,131**	
EM	6 w/o	Cutoff Wall (Embankment Area)	3,484,800	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	8,712,000	0	8,712,000	(Soil Bentonite)
SC	6 w/o	Cutoff Wall (Through Limestone)	25,555	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	1,149,984	0	1,149,984	39,210,544
EM	9	Dozer and Dump Trucks (Select Fil	1,187,155	CY	0.74	C6	0.024	27,898	996.4	14.17	395,316.75	0.00	0.00	3f1	2	60.38	481,299.85		0	876,617	
EM	9	Dozer Angle Blade (Select Fill Belo	1,187,155	CY	0.69	B2	0.009	10,684	667.8	16.37	174,903.58	0.00	0.00	6c	1	120.02	641,189.84		0	816,093	
EM	9	Compact (Select Fill Below Drain)	1,187,155	CY	0.58	B5	0.009	10,684	890.4	17.22	184,020.93	0.00	0.00	6g1	1	70.54	502,463.37		0	686,484	3,561,466
EM	8	Dozer Loader (Rock Fill Borrow Are	1,788,864	CY	1.15	B2	0.015	26,833	1,677.1	16.37	439,255.56	0.00	0.00	6c	1	120.02	1,610,294.12		0	2,049,550	
EM	8	Dump Truck (Rock Fill Borrow Are	2,504,410	CY	0.93	C1	0.009	22,540	2,817.5	11.88	267,771.47	0.00	0.00	3e1	1	91.31	2,058,185.61		0	2,325,957	
EM	8	Dozer Angle Blade (New Embankm	2,504,410	CY	1.03	B2	0.014	33,810	2,113.1	16.37	553,462.00	0.00	0.00	6c	1	120.02	2,028,970.59		0	2,582,433	6,957,939
EM	10	Common Borrow (Top Soil - Peat)	0	CY	0.00	C6	0.047	0	0.0	14.17	0.00	0.00	0.00	3f	3	63.57	0.00		0	0	0
	9	Fine Grading	0	SY	0.00	B5a	0.008	0	0.0	17.30	0.00	0.00	0.00	7d	1	22.97	0.00		0	0	
02370		**Erosion and Sedimentation Control**																			
		Rip Rap																			
EM	8	12' Rad. Crawler Mtd. (Prod. Blast	0	CY	0.00	B2	0.258	0	0.0	16.37	0.00	18.75	0.00	4b	2	26.80	0.00		0	0	
EM	8	Hauling (5 Miles)	0	CY	0.00	C1	0.040	0	0.0	11.88	0.00	0.00	0.00	3e	1	49.87	0.00		0	0	0
02700		**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o	Crusher (Transition Borrow Area)	*64533*	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		806,667	806,667	
EM	8 w/o	Aggregate Base 1-1/2" Stone, 12"	*193600*	SY	0.91	B7a	0.017	3,291	51.4	16.83	55,386.78	0.00	0.00	7a+	6	36.88	121,384.06		0	176,771	983,438
		Subtotal Site Construction						1,103,769			$17,690,904		$20,156,393				$60,581,541	$19,154,784	$5,932,582	$123,516,205	
3		**Concrete**																			
03050		**Basic Concrete Materials and Methods**																			
03100		**Concrete and Forms and Accessories**																			
03200		**Concrete Reinforcement**																			
03300		**Cast-In-Place Concrete**																			
03310		**Structural Concrete**																			
EM	11	Parapet Wall Footing	0	CY	0.00	D8b	1.382	0	0.0	17.59	0.00	199.85	0.00	8b	3	8.93	0.00		0	0	
EM	11	Parapet Wall	51,110	CY	174.72	D8b	2.182	111,523	1,742.5	17.59	1,961,408.88	116.85	5,972,250.24	8b	3	8.93	996,231.63		0	8,929,891	
EM	**11**	**Concrete Batch Plant and Delive**	**52,644**	**CY**	**26.15**	**C6a**	**0.480**	**25,269**	**394.8**	**14.44**	**364,757.75**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,011,715.42**		**0**	**1,376,473**	$201.65
03370		**Specially Placed Concrete**																			
		Roller Compacted Concrete																			
EM	11	Mass Placement, 1' Lift, 12" Layer	1350941	CY	1.07	B5	0.009	12,158	506.6	16.91	205,559.15		0.00	6f1	2	102.16	1,242,085.96		0	1,447,645	
EM	11	Vertical Face, Formed, 1' Lift	214896	CY	7.14	B5	0.060	12,894	537.2	16.91	217,990.50		0.00	6f1	2	102.16	1,317,202.08		0	1,535,193	
EM	11	Vertical Face, Forms (4 Uses)	3484800	SF	4.75	C5b	0.127	440,827	7,871.9	16.25	7,161,552.74	1.19	4,146,912.00	5a1	2	11.91	5,250,584.09		0	16,559,049	
EM	11	Sloped Face, Non-formed, 1' Lift	428630	CY	5.00	B5	0.042	18,002	750.1	16.91	304,361.87		0.00	6f1	2	102.16	1,839,098.90		0	2,143,461	

EAA Reservoir A-1 Basis of Design Report

January, 2006

Case: 8

Opinion Of Probable Cost

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

EAA Reservoir A-1, RCC (OG+23) "Including 15 Foot Wave Break Bench OG +16", With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total Linear Feet

						Labor						Material		Equipment							
CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost	Remarks
EM	11	Roller Compacted Concrete, 1.5"-2	1994467	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	89,751,024				0.00		0	89,751,024	100 lbs of cement by volume
EM	11	Dump Truck (18 CY) Conveying Ma	1994467	CY	1.24	C1	0.012	23,934	2,991.7	11.88	284,331.24		0.00	3e1	1	91.31	2,185,469.82		0	2,469,801	(15 Min. Cycles)
EM	11	Truck Mtd. Hydraulic Crane 100 To	1565837	CY	0.59	D18a	0.025	39,146	978.6	17.43	682,469.97		0.00	8g	4	6.32	247,243.12		0	929,713	
EM	11	Surface Prep. Vacuum Truck	0	SY	0.00	C6a-	0.006	0	0.0	17.72	0.00		0.00	3j2	1	12.86	0.00		0	0	
EM	11	Surface Prep. Water Blast	0	SY	0.00	C1a	0.030	0	0.0	15.63	0.00		0.00	3j4	5	13.25	0.00		0	0	$57.58
EM	**11**	**Concrete Batch Plant and Delive**	**2054301**	**CY**	**26.15**	**C6a**	**0.480**	**986,065**	**15,407.3**	**14.44**	**14,233,842.27**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**39,479,895.01**		**0**	**53,713,737**	$84.51
		Subtotal Concrete						1,669,818			$25,416,274		$99,870,186				$53,569,526	$0	$0	$178,855,987	
		Construction Subtotal (Direct Costs)						2,773,587			$43,107,179		$120,026,579				$114,151,067	$19,154,784	$49,776,550	**$346,216,159**	

Indirect Costs				
Sales Tax	6%	of purchased materials + Rental Equipment		14,050,659
Overhead and Profit	16%	of construction cost + general requirements	1.000000	56,486,981
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit		14,586,383
Project Reserve	5%	of construction cost		21,567,009
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation		135,872,157
Construction Subtotal Indirects				**$242,563,189**
Total Construction (Directs and Indirect Costs)				**$588,779,348**
Permits				0
Design	0%	of construction cost		0
Construction Management	0%	of construction cost		0
Total				**$588,779,348**

TABLE 8.13-9

OPINION OF PROBABLE CONSTRUCTION COST FOR CASE 9

Case: 9

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02

Project No.: 141731.0910

Revision No.: 2

Date: 07/27/05

Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG+25) "Including 5 Foot Parapet Wall," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials and Methods				55,465	951,904	0	6,923,738	0	0	7,875,642	0.8732	6,877,052	14,752,694
	Subtotal				55,465	$951,904	$0	$6,923,738	$0	$0	$7,875,642		$6,877,052	$14,752,694
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials and Methods				186,264	3,059,757	2,297,434	11,810,334	0	0	17,167,525	0.8732	14,990,774	32,158,299
	Subtotal				186,264	$3,059,757	$2,297,434	$11,810,334	$0	$0	$17,167,525		$14,990,774	$32,158,299
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials and Methods				37,359	541,910	0	3,562,699	0	0	4,104,608	0.8732	3,584,166	7,688,774
	Subtotal				37,359	$541,910	$0	$3,562,699	$0	$0	$4,104,608		$3,584,166	$7,688,774
4	**Embankment Construction (Production Blast Cap Rock and Excavate Silty Sand, etc.)**													
	Materials and Methods				76,159	1,243,513	858,610	4,765,586	0	0	6,867,710	0.8732	5,996,921	12,864,630
	Subtotal				76,159	$1,243,513	$858,610	$4,765,586	$0	$0	$6,867,710		$5,996,921	$12,864,630
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials and Methods				0	0	0	0	0	0	0	0.8732	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
6	**Embankment Construction (Surface Preparation / Cutoff Wall)**													
	Materials and Methods				204,048	3,203,143	12,262,624	4,591,000	97,562,784	0	117,619,551	0.8732	102,706,018	220,325,568
	Subtotal				204,048	$3,203,143	$12,262,624	$4,591,000	$97,562,784	$0	$117,619,551		$102,706,018	$220,325,568
7	**Embankment Construction (Sand filters and Drains)**													
	Materials and Methods				0	0	0	0	0	0	0	0.8732	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
8	**Embankment Construction (Rock Fill)**													
	Materials and Methods				3,291	55,387	0	121,384	0	806,667	983,438	0.8732	858,743	1,842,180
	Subtotal				3,291	$55,387	$0	$121,384	$0	$806,667	$983,438		$858,743	$1,842,180
9	**Embankment Construction (Broken Cap Rock And Silty Sand, Shell, Etc. Soils)**													
	Materials and Methods				0	0	0	0	0	0	0	0.8732	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials and Methods				0	0	0	0	0	0	0	0.8732	0	0
	Subtotal				0	$0	$0	$0	$0	$0	$0		$0	$0
11	**Embankment Construction (RCC And Concrete)**													
	Materials and Methods				1,782,030	27,644,696	83,376,628	46,812,204	0	0	157,833,528	0.8732	137,821,077	295,654,605
	Subtotal				1,782,030	$27,644,696	$83,376,628	$46,812,204	$0	$0	$157,833,528		$137,821,077	$295,654,605
12	**Equipment Mobilization Or Demobilization**													
	Materials and Methods				737	9,103	0	43,887	0	87,069	140,059	0.8732	122,300	262,359
	Subtotal				737	$9,103	$0	$43,887	$0	$87,069	$140,059		$122,300	$262,359
	Total				**2,345,354**	**$36,709,412**	**$98,795,296**	**$78,630,832**	**$97,562,784**	**$893,736**	**$312,592,060**		**$272,957,050**	**$585,549,110**

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

Case: 9
Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG+25) "Including 5 Foot Parapet Wall," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total LF

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor: Crew Code	Labor: M-H per Unit	Labor: Man-Hours	Labor: Duration Days	Labor: Average Wage Rate	Labor: Labor Cost	Material: Unit Cost	Material: Material Cost	Equipment: Code	Equipment: No.	Equipment: Avg. Cost ($/hr)	Equipment: Equipment Cost	Sub-contract	Other	Total Cost	Remarks
1	**General Requirements**																			
	Mobilization	1	LS	3,125,921						0.00		0.00						3,125,921	3,125,921	
	Supervision	1	LS	15,629,603						0.00		0.00						15,629,603	15,629,603	
	Temporary Construction Facilities	1	LS	7,814,801						0.00		0.00						7,814,801	7,814,801	
	Temporary Utilities	1	LS	4,688,881						0.00		0.00						4,688,881	4,688,881	
	Safety	1	LS	7,814,801						0.00		0.00						7,814,801	7,814,801	
	Miscellaneous	1	LS	6,251,841						0.00		0.00						6,251,841	6,251,841	
	Subtotal Mobilization									$0		$0				$0	$0	$45,325,849	$45,325,849	
2	**Site Work**																			
02225	**Demolition**																			
02230	**Site Clearing**																			
SC	1 Scraper (Strip Peat Canal Area)	860,444	CY	2.06	B5b	0.012	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
SC	1 Scraper (Strip Peat Bench Area)	1720889	CY	1.55	B5b	0.009	15,488	774.4	17.17	265,867.01	0.00	0.00	1s1	2	154.87	2,398,662.81		0	2,664,530	
EM	1 Scraper (Strip Peat Embankment A	1010592	CY	1.03	B5b	0.006	6,064	303.2	17.17	104,086.93	0.00	0.00	1s1	2	154.87	939,076.49		0	1,043,163	
EM	1 Surface Prep. (Strip Peat Embankn	1515888	SY	0.35	B5	0.008	12,127	1,010.6	17.22	208,869.15		0.00	3j4+	2	26	319,624.17		0	528,493	
EM	1 Scraper (Strip Peat Inside Bench A	2581333	CY	0.69	B5b	0.004	10,325	516.3	17.17	177,244.67	0.00	0.00	1s1	2	154.87	1,599,108.54		0	1,776,353	
EM	1 Dozer Angle Blade (Strip Peat Insic	75715	CY	1.15	B2	0.015	1,136	71	16.37	18,592	0.00	0.00	6c	1	120.02	68,157		0	86,749	
02240	**Dewatering**																			
	Seepage Canal																			
SC	2 Pump,12" Suction (Make-up)	38,914	LF	40.86	B4	0.040	1,557	194.6	16.87	26,255.00	0.00	0.00			40.19	1,563,780.00		0	1,590,035	
SC	2 12" Dia Pipe 2,500 GPM	54,595	LF	5.48	B4	0.098	5,350	668.8	16.87	90,246.68	0.00	0.00			3.83	209,099.62		0	299,346	
	Production Blast																			
SC	4 Pump,12" Suction (Make-up)	26136	LF	40.86	B4	0.040	1,045	130.7	16.87	17,633.95	0.00	0.00			40.19	1,050,300.00		0	1,067,934	
SC	4 12" Dia Pipe 2,500 GPM	92928	LF	5.48	B4	0.098	9,107	1,138.4	16.87	153,611.38	0.00	0.00			3.83	355,914.24		0	509,526	
	Cutoff Wall																			
SC	2 w/o Pump,12" Suction (Make-up)	*31944*	LF	40.86	B4	0.040	1,278	159.7	16.87	21,552.61	0.00	0.00			40.19	1,283,700.00		0	1,305,253	
SC	2 w/o 12" Dia Pipe 2,500 GPM	*72,019*	LF	5.48	B4	0.098	7,058	882.2	16.87	119,048.82	0.00	0.00			3.83	275,833.54		0	394,882	
02300	**Earthwork**																			
02305	**Equipment Mobilization Or Demobilization**																			
	12 Dump Truck (26 Tons)	12	EA	346	C1	2.000	24	3.0	11.88	285.12	0.00	0.00	3e1	1	49.87	1,196.96		2,668	4,150	Other: Driving Cost for 140Miles
	12 Dozers (Above 150 HP)	28	EA	578	C1	2.667	75	9.3	11.88	887.15	0.00	0.00	3g	2	65.48	4,889.41		10,398	16,174	Other: Driving Cost for 140Miles
	12 Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242	Other: Driving Cost for 140Miles
	12 Crawler Type Drill, 4"	16	EA	322	C2	6.000	96	12.0	15.55	1,492.80	0.00	0.00	6h	4	34.56	3,318.19		346	5,157	Other: Driving Cost for 140Miles
	12 Air Compressor, 600 CFM	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 50 Ft Air Hose, 3" Dia.	16	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0	Included Drilling Machine
	12 Excavator, Diesel Hydraulic, Crawl	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12 Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Concrete Batch Plant, Portable, 2	**8**	**EA**	**3,466**	**C1**	**16.000**	**128**	**16.0**	**11.88**	1,520.64	**0.00**	**0.00**	3g	2	65.48	8,380.80		**17,823**	27,724	Other: Driving Cost for 140Miles
	12 Concrete Transit Mixer Truck	**24**	**EA**	**461**	**C1**	2.667	**64**	**8.0**	**11.88**	760.42	**0.00**	**0.00**	3e1	1	49.87	3,192.29		**7,115**	11,068	Other: Driving Cost for 140Miles
	12 Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155	Other: Driving Cost for 140Miles
	12 Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777	Other: Driving Cost for 140Miles
	12 Truck Mtd. Crane up To 75 Ton	12	EA	481	C1	2.222	27	3.3	11.88	316.77	0.00	0.00	3g	2	65.48	1,745.83		3,713	5,775	Other: Driving Cost for 140Miles
	12 Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090	Other: Driving Cost for 140Miles
	12 Attachment Clam Bucket, 1/2 CY	12	EA	578	C1	2.667	32	4.0	11.88	380.21	0.00	0.00	3g	2	65.48	2,095.46		4,456	6,932	Other: Driving Cost for 140Miles
	12 Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
	12 Wheeled Skid Steer, Diesel, w/ Bro	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087	Other: Driving Cost for 140Miles
	12 Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311	Other: Driving Cost for 140Miles
	12 Wash and Screen (Sand Horiz. Bla	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621	Other: Driving Cost for 140Miles
02310	**Grading**																			
02315	**Excavation and Fill**																			
SC	2 Drilling and Blasting (Seepage Can	**1,215,573**	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887	
SC	2 Excavating Cap Rock (Seepage Ca	**1,215,573**	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330	
SC	2 Dump Truck (Canal/Stock Pile Area	**1,519,467**	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973	(Avg. 1,000 FT. Around Trip)
SC	2 Hoe Ram (Stock Pile Area)	**159,544**	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879	
SC	2 Dozer Angle Blade (Stock Pile Area	**1,519,467**	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939	13,578,008 ***17,167,525***
SC	3 Excavated - Silty, Sand, Shells (Se	**958,372**	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786	
SC	3 Cut Through Limestone	**193,406**	CY	4.02	B3	0.025	4,835	201.5	16.75	80,988.93	0.00	0.00	1n	1	144.10	696,761.77		0	777,751	
SC	3 Haul to Dewater and Work Stock P	**1,266,956**	CY	1.33	C1	0.013	16,330	2,041.2	11.88	193,996.33	0.00	0.00	3e1	1	91.31	1,491,123.94		0	1,685,120	(Avg. 1,000 FT. Around Trip)
SC	3 Hauling (5 Miles)	0	CY	0.00	C1	0.040	0	0.0	11.88	0.00	0.00	0.00	3e	1	49.87	0.00		0	0	
SC	3 Dozer Angle Blade - Work Stock Pi	**1,266,956**	CY	0.69	B2	0.009	11,403	712.7	16.37	186,660.65	0.00	0.00	6c	1	120.02	684,290.84		0	870,951	***4,104,608***
EM	4 Drilling and Blasting (Prod. Blast Ar	**454,291**	CY	6.02	B6	0.080	36,343	1,514.3	17.11	621,954.94	1.89	858,610.37	6h	4	34.56	1,256,187.11		0	2,736,752	
EM	4 Excavating Cap Rock (Prod. Blast /	**454,291**	CY	1.75	B2	0.015	6,718	419.9	16.37	109,970.90	0.00	0.00	1b	1	102.03	685,441.23		0	795,412	
EM	4 Dump Truck (Prod. Blast Stock Pile	**567,864**	CY	1.71	C1	0.017	9,410	1,176.3	11.88	111,794.57	0.00	0.00	3e1	1	91.31	859,292.37		0	971,087	(Avg. 2 Miles Around Trip)
EM	4 Hoe Ram (Stock Pile Area)	**59,626**	CY	1.74	A10	0.077	4,591	459.1	17.89	82,136.22	0.00	0.00	6i	1	5.91	21,714.45		0	103,851	
EM	4 Dozer Angle Blade (Prod. Blast Are	**567,864**	CY	1.20	B2	0.016	8,944	559.0	16.37	146,410.96	0.00	0.00	6c	1	120.02	536,736.98		0	683,148	5,290,250
EM	6 Clean Cap Rock Surface (Embankı	**298,531**	SY	0.87	C1a	0.030	8,956	373.2	15.63	140,011.13		0.00	3j4	5	13.25	118,655.41		0	**258,667**	
EM	6 Cement Grout Cap Rock (Embankı	**48,787**	CY	78.98	B4	1.500	73,181	2,286.9	16.87	1,234,377.14	45.00	2,195,424.00	3i	2	5.78	423,183.35		0	**3,852,984**	
EM	**6 Concrete Batch Plant and Delive**	**50,251**	**CY**	**26.15**	**C6a**	**0.480**	**24,120**	**376.9**	**14.44**	**348,177.85**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**965,728.36**		**0**	**1,313,906**	

Project: Evaluate and Select Alternative Embankment Section, Work Order NO. CN040932-WO02
Project No. 141731.0910
Revision No.: 2
Date: 07/27/05

Case: 9
Opinion Of Probable Cost

EAA Reservoir A-1, RCC (OG+25) "Including 5 Foot Parapet Wall," With 30 Foot Cutoff Wall, and Seepage Canal 15 Feet Deep

22 Miles = 116,160 Total LF

CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Labor						Material		Equipment				Sub-contract	Other	Total Cost	Remarks
					Crew Code	M-H per Unit	Man-Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost				
EM	6 w/o Cutoff Wall (Embankment Area)	116,160	LF	80.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	9,292,800	0	9,292,800	(Quoted for 2' Wide at $40/LF)
EM	6 w/o Lean Concrete Fill In Cap Rock Cu	154,880	CY	67.54	D8	0.137	21,219	442.1	17.69	375,250.23	65.00	10,067,200.00	8b1	2	0.83	17,628.35		0	10,460,079	
EM	**6 w/o Concrete Batch Plant and Delive**	**159,526**	**CY**	**26.15**	**C6a**	**0.480**	**76,573**	**1,196.4**	**14.44**	**1,105,326.52**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**3,065,804.31**		**0**	**4,171,131**	
EM	6 w/o Cutoff Wall (Embankment Area)	3,484,800	SF	25.00	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	87,120,000	0	87,120,000	(Plastic Concrete)
SC	6 w/o Cutoff Wall (Through Limestone)	25,555	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	1,149,984	0	1,149,984	*117,619,551*
02700	**Bases, Ballasts, Pavements and Appurtenances**																			
EM	8 w/o Crusher (Transition Borrow Area)	*64533*	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		806,667	806,667	
EM	8 w/o Aggregate Base 1-1/2" Stone, 12" '	*193600*	SY	0.91	B7a	0.017	3,291	51.4	16.83	55,386.78	0.00	0.00	7a+	6	36.88	121,384.06		0	176,771	983,438
	Subtotal Site Construction						563,324			$9,064,717		$15,418,668				$31,818,627	$97,562,784	$893,736	$154,758,532	
3	**Concrete**																			
03050	**Basic Concrete Materials and Methods**																			
03100	**Concrete and Forms and Accessories**																			
03200	**Concrete Reinforcement**																			
03300	**Cast-In-Place Concrete**																			
03310	**Structural Concrete**																			
EM	11 Parapet Wall Footing	0	CY	0.00	D8b	1.382	0	0.0	17.59	0.00	199.85	0.00	8b	3	8.93	0.00		0	0	
EM	11 Parapet Wall	51,110	CY	174.72	D8b	2.182	111,523	1,742.5	17.59	1,961,408.88	116.85	5,972,250.24	8b	3	8.93	996,231.63		0	8,929,891	
EM	**11 Concrete Batch Plant and Delive**	**52,644**	**CY**	**26.15**	**C6a**	**0.480**	**25,269**	**394.8**	**14.44**	**364,757.75**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**1,011,715.42**		**0**	**1,376,473**	$201.65
03370	**Specially Placed Concrete**																			
	Roller Compacted Concrete																			
EM	11 Mass Placement, 1' Lift, 12" Layer	1321901	CY	1.07	B5	0.009	11,897	495.7	16.91	201,140.43		0.00	6f1	2	102.16	1,215,385.92		0	1,416,526	
EM	11 Vertical Face, Formed, 1' Lift	232320	CY	7.14	B5	0.060	13,939	580.8	16.91	235,665.41		0.00	6f1	2	102.16	1,424,002.25		0	1,659,668	
EM	11 Vertical Face, Forms (4 Uses)	6272640	SF	4.75	C5b	0.127	793,489	14,169.4	16.25	12,890,794.93	1.19	7,464,441.60	5a1	2	11.91	9,451,051.36		0	29,806,288	
EM	11 Sloped Face, Non-formed, 1' Lift	0	CY	0.00	B5	0.042	0	0.0	16.91	0.00		0.00	6f1	2	102.16	0.00		0	0	
EM	11 Roller Compacted Concrete, 1.5"-2	1554221	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	69,939,936				0.00		0	69,939,936	100 lbs of cement by volume
EM	11 Dump Truck (18 CY) Conveying Ma	1554221	CY	1.24	C1	0.012	18,651	2,331.3	11.88	221,569.72		0.00	3e1	1	91.31	1,703,062.68		0	1,924,632	(15 Min. Cycles)
EM	11 Truck Mtd. Hydraulic Crane 100 To	1554221	CY	0.59	D18a	0.025	38,856	971.4	17.43	677,407.14		0.00	8g	4	6.32	245,408.97		0	922,816	
EM	11 Surface Prep. Vacuum Truck	0	SY	0.00	C6a-	0.006	0	0.0	17.72	0.00		0.00	3j2	1	12.86	0.00		0	0	
EM	11 Surface Prep. Water Blast	0	SY	0.00	C1a	0.030	0	0.0	15.63	0.00		0.00	3j4	5	13.25	0.00		0	0	$67.99
EM	**11 Concrete Batch Plant and Delive**	**1600847**	**CY**	**26.15**	**C6a**	**0.480**	**768,407**	**12,006.4**	**14.44**	**11,091,951.63**	**0.00**	**0.00**	**8h1**	**6**	**40.04**	**30,765,346.26**		**0**	**41,857,298**	$94.92
03400	**Precast Concrete**																			
03500	**Cementitious Decks and Underlay**																			
03600	**Grouts**																			
03900	**Concrete Restorations and Cleaning**																			
		0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0	*147,527,164*
	Subtotal Concrete						1,782,030			$27,644,696		$83,376,628				$46,812,204	$0	$0	$157,833,528	
	Construction Subtotal (Direct Costs)						2,345,354			$36,709,412		$98,795,296				$78,630,832	$97,562,784	$46,219,584	**$357,917,908**	

Indirect Costs

Sales Tax	6%	of purchased materials + Rental Equipment	10,645,568
Overhead and Profit	13%	of construction cost + general requirements	45,903,874
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	14,506,357
Project Reserve	5%	of construction cost	21,448,685
Contingency	30%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	135,126,718
Construction Subtotal Indirects			**$227,631,202**
Total Construction (Directs and Indirect Costs)			**$585,549,110**
Permits			0
Design	0%	of construction cost	0
Construction Management	0%	of construction cost	0
Total			**$585,549,110**

SECTION 9

RESERVOIR SEEPAGE

TABLE OF CONTENTS

TABLES

FIGURES

9. RESERVOIR SEEPAGE

9.1 INTRODUCTION

This Section of the BODR describes the methods to determine EAA Reservoir A-1 seepage and flows in the seepage canals. As with other surface water features such as STAs and canals, seepage will occur from EAA Reservoir A-1 because the soil within approximately 200 feet below the surface of the site is highly permeable. Changes to existing groundwater flow patterns beneath the EAA Reservoir A-1 will be caused by seepage from the EAA Reservoir A-1 and how the seepage controls are built and operated. Changes to groundwater flow beneath the EAA Reservoir A-1 will also be caused by the elimination of farmland within the footprint of the EAA Reservoir A-1, which will alter the groundwater control pattern. The goal of seepage control is to minimize or eliminate the impacts from changes to groundwater flow and groundwater levels in areas surrounding the EAA Reservoir A-1.

Both two-dimensional and three-dimensional modeling of potential impacts to groundwater from the construction of the EAA Reservoir A-1 were performed to evaluate seepage for a large number of combinations of EAA Reservoir A-1 water depth; cutoff wall depth and location; seepage canal depth, location, and operation; and other seepage control alternatives such as pressure-relief wells. The computer models provide a great deal of assistance in understanding the interaction of the proposed EAA Reservoir A-1 with the surrounding areas and provide direction in designing the EAA Reservoir A-1 and associated facilities.

Both three-dimensional MODFLOW groundwater modeling and two-dimensional SEEP/W groundwater modeling were performed to analyze seepage from EAA Reservoir A-1. Aquifer parameters used in the EAA Reservoir A-1 seepage models were determined from calibration of the models against the results of the Test Cell Project. The calibration of horizontal and vertical conductivity (K_H and K_V, respectively) is described in Section 2.6. Appendices 9-1 and 9-2 describe the development of the groundwater models of EAA Reservoir A-1 in more detail.

The groundwater models were used to evaluate the following major issues:

- The effect of seepage on embankment stability
- The amount of water the EAA Reservoir A-1 loses to seepage
- The percentage of seepage that is collected and returned to the EAA Reservoir A-1
- The effectiveness of various seepage control alternatives
- The amount of unrecoverable seepage, if any, that migrates to surrounding areas for the various seepage control alternatives
- The effect of any unrecoverable seepage on groundwater levels in the surrounding areas

The surrounding areas include 1) farmland to the north and west of the EAA Reservoir A-1 and to the east of the NNRC, 2) U.S. 27 immediately east of the EAA Reservoir A-1, 3) STA-3/4 to the south of the EAA Reservoir A-1, and 4) the Holey Land to the southwest of the EAA Reservoir A-1. Goals for managing seepage to each of these areas are as follows:

- *Farmland.* Control groundwater levels to prevent impact to crops from seepage and to prevent the need for more than normal pumping from the farm canals surrounding the EAA Reservoir A-1.
- *U.S. 27.* Control seepage to prevent groundwater levels from rising into the base of the highway or into the adjacent drainage ditches (Per Florida Department of Transportation).
- *STA-3/4.* Control seepage to STA-3/4 and to the Supply Canal (which is released to STA-3/4) to an acceptable percentage of the capacity of STA-3/4. If an acceptable percentage cannot be defined, there may be the need to eliminate seepage to STA-3/4 at an added cost to the Project.
- *Holey Land.* Control seepage to the Holey Land to an amount that can be offset by a reduction in surface water deliveries such that the Holey Land can be maintained at its targeted water level.

Definitions of “impacts” to the surrounding areas caused by seepage can be rather subjective. To simplify this definition, it is assumed that any change in groundwater levels less than the predictive accuracy of the groundwater computer model is considered “no impact.” For the purpose of defining “impact,” we have chosen any change greater than 0.3 feet of increase in the groundwater level, as causing an impact.

9.2 GROUNDWATER MODEL RESULTS

9.2.1 Seepage Quantities

Groundwater modeling was used to evaluate a large number of scenarios of EAA Reservoir A-1 design, including water depth, cutoff wall depth, and seepage canal depth. The total seepage rates from EAA Reservoir A-1 for selected scenarios are shown in Table 9.2-1. The rates given in Table 9.2-1 are based on steady-state MODFLOW model runs where the water levels in the EAA Reservoir A-1, the surrounding areas, and canals are held constant, allowing the groundwater to reach equilibrium. The results indicate seepage is more sensitive to the depth of the cutoff wall than the depth of the seepage canal bottom. Therefore, a seepage canal depth of 10 feet was selected.

Table 9.2-1 EAA Reservoir A-1 Seepage

EAA Reservoir A-1 Depth (feet)	**Cutoff Wall Depth** (feet)	**Seepage Canal Depth** (feet)	**Total EAA Reservoir A-1 Seepage** (cubic feet per second)
1	34	10	41
1	34	20	44
1	69	10	15
1	69	20	16
3	34	10	89
3	34	20	93
3	69	10	46
3	69	20	47
6	34	10	160
6	34	20	167
6	69	10	92
6	69	20	94
12	34	10	303
12	34	20	314
12	69	10	185
12	69	20	188

The results given in Table 9.2-1 represent seepage rates expected from the EAA Reservoir A-1 assuming the EAA Reservoir A-1 water depth remains constant. In reality, the water levels in the EAA Reservoir A-1 and surrounding areas will be highly variable, and seepage rates of this magnitude will not occur all of the time. Additional groundwater modeling was performed to evaluate monthly changes to EAA Reservoir A-1 water levels, and the water levels in the surrounding areas and canals. The computer model results provide a better assessment of annual seepage volume as shown in Table 9.2-2 for a variety of cutoff wall configurations. Table 9.2-2 also provides a comparison of filling the EAA Reservoir A-1 to a depth of 12 feet using only the northeast pump station, versus allowing the EAA Reservoir A-1 to be filled to a depth of eight feet by releasing water from the Supply Canal through gates, then closing the gates and allowing the EAA Reservoir A-1 to be filled from a depth of eight feet to a depth of 12 feet by the northeast pump station. The latter condition is referred to as “floating” the EAA Reservoir A-1 and the Supply Canal together. Floating the EAA Reservoir A-1 and the Supply Canal together provides significant cost savings for the EAA Reservoir A-1 Project, as described elsewhere in this report, since modifications to G-370 and G-372 pump stations would not be required for this option.

Table 9.2-2 Total Annual Seepage from EAA Reservoir A-1 and Flow Collected by Seepage Canal*

Cutoff Wall Depths (feet)			Float EAA Reservoir A-1 and Supply Canal Together to Eight Foot depth?	Total Seepage from EAA Reservoir A-1* (acre-feet per year)	Flow Collected by Seepage Canal** (acre-feet per year)
North and East Sides	West Side Along Future EAA Reservoir A-2 Site	Along Holey Land and STA-3/4			
34	34	10	No	165,299	121,509
34	34	69	No	144,700	120,847
69	34	10	No	110,971	80,344
69	34	69	No	90,014	79,501
34	34	10	Yes	159,733	121,872
34	34	69	Yes	141,796	121,269
69	34	10	Yes	105,350	80,711
69	34	69	Yes	87,110	79,864

* Annual flows calculated from average monthly model results
** Not all of the flow collected by the Seepage Canal is from EAA Reservoir A-1 seepage. A portion of the flow comes from the NNRC

9.2.2 Cutoff Wall Depth

The results provided in Tables 9.2-1 and 9.2-2 were used to assist in determining the optimum cutoff wall depth. Along the north, west, and east sides of the EAA Reservoir A-1, the minimum cutoff wall depth is the base of the Fort Thompson Formation (34 feet below land surface) to ensure embankment stability. Along the south side of the EAA Reservoir A-1, a shallower cutoff wall would maintain stability because the water levels in the Supply Canal and STA-3/4 provide a balancing hydraulic head against seepage. Additional discussion of the requirement of cutoff walls for embankment stability is provided in Section 8 of this report.

A present worth (PW) analysis was performed using the modeled seepage rates for 34-foot (bottom of Fort Thompson Formation) and 69-foot (bottom of Caloosahatchee Formation) cutoff wall depths along the northwest, north, and east sides of the EAA Reservoir A-1. This PW analysis included capital cost associated with installing the cutoff wall; purchasing and installing pumps and engine driven generators to capture and return seepage to the EAA Reservoir A-1; replacing the pumps and engine driven generators periodically over the 50-year design life; and O&M of equipment over the 50-year period, the latter including fuel costs. The results of this analysis showed that there was minimal difference between PW of the 34-foot and 69-foot cutoff alternatives; or more aptly, it took most of the 50-year design life to recoup the extra capital cost spent to install the deeper cutoff wall. Therefore, since initial savings in capital cost is preferred, the 34-foot cutoff was selected for the northwest, north, and east sides of the EAA Reservoir A-1.

Along the south side of the EAA Reservoir A-1 adjacent to STA-3/4 and the Holey Land, cutoff wall depths of 10 feet, 34 feet, and 69 feet were evaluated. A 10 foot cutoff wall on the south side would be the most cost-effective and is the minimum depth required for embankment stability. However, additional cutoff wall depth and potentially other seepage control measures could be required to reduce seepage that migrates to STA-3/4 and the Holey Land, as described in the seepage control alternatives in the following section.

The depths of 10, 34, and 69 feet in the groundwater models were estimated based on available data from previously drilled test holes across the EAA Reservoir A-1 Project area. At the Test Cell site, the depth to the bottom of the Fort Thompson Formation was closer to 30 feet, as described in Section 8. It is anticipated that the actual depth of the cutoff wall will vary depending on the aquifer thickness encountered during construction. Additional test drilling is currently being performed to determine depths of the aquifer formations beneath the proposed location of the EAA Reservoir A-1 embankment.

9.3 SEEPAGE CONTROL ALTERNATIVES

Although quite effective at reducing seepage, cutoff walls constructed to these depths cannot completely eliminate seepage from EAA Reservoir A-1. Additional seepage control measures were considered, including the effect of lowering the water level in the seepage canal as a way to draw seepage to the surface, then use pressure-relief wells to intercept deep seepage before it migrates to surrounding areas. Five seepage control alternatives were evaluated with MODFLOW, as described below. The model results for these alternatives are based on the assumption that the agricultural canals and control structure in the surrounding areas such as the farms, STA-3/4, and the Holey Land, would not be operated to offset the rise in groundwater levels in the agricultural areas.

9.3.1 Seepage Control Alternative 1

This alternative includes seepage controls that are required to essentially eliminate impacts to surrounding farms, U.S. 27, the Holey Land, and STA-3/4. In this alternative, a cutoff wall depth of 34 feet is provided around the entire EAA Reservoir A-1. A seepage canal with a bottom depth of 13.5 feet surrounds the entire EAA Reservoir A-1, and the water level in the seepage canal is held 3.5 feet lower than the water level in the surrounding farmlands. The water collected by the seepage canal is pumped back into the EAA Reservoir A-1. Groundwater model results indicate the total seepage from EAA Reservoir A-1 is approximately 341 cfs at an average depth of 8.8 feet based on the 35 year period of simulation from the water balance model. This is equivalent to approximately 246,900 acre-feet per year. The model indicates 433 cfs is pumped from the seepage canal and returned to the EAA Reservoir A-1 at the average EAA Reservoir A-1 depth, which is more than the total seepage from the EAA Reservoir A-1. This is because the seepage canal collects not only seepage but also induces flow from the Supply Canal and NNRC since it is held lower than the levels in these canals. In addition, there is an existing seepage canal along the north side of the Supply Canal, which is adjacent to the Holey Land. This seepage canal controls seepage into the farmland north of the Holey Land. (Figures 9.3-1 through 9.3-8 are at the back of this section.)

Figure 9.3-1 shows the layout for this alternative and shows the increase in groundwater head in surrounding areas will be less than 0.3 feet at an average EAA Reservoir A-1 depth of 8.8 feet.

Figure 9.3-2 shows the effect on groundwater heads if the EAA Reservoir A-1 depth is maintained at 12 feet for a long period of time.

9.3.2 Seepage Control Alternative 2

This alternative includes a 34-foot deep cutoff wall and a seepage canal with a bottom depth of 10 feet around the northwest, north, and east sides of the EAA Reservoir A-1. There is a 10-foot deep cutoff wall and no seepage canal along STA-3/4 and the Holey Land. The water level in seepage canal is held the same as the water level in the surrounding farmlands, and the water collected by the seepage canal is pumped back into the EAA Reservoir A-1. In addition, there is an existing seepage canal along the north side of the Supply Canal, which is adjacent to the Holey Land. This seepage canal controls seepage into the farmland north of the Holey Land. Groundwater model results indicate that the total seepage from EAA Reservoir A-1 is approximately 230 cfs at an average EAA Reservoir A-1 depth of 8.8 feet based on the 35 year period of simulation from the water balance model. The model estimates that 174 cfs is collected by the seepage canal for this alternative. Approximately 13 cfs will migrate to the farmlands, which is equivalent to 9,400 acre-feet per year. This additional water would be pumped through the farm canals back to the major canals in order to maintain water levels in the farmlands at their current levels, or the groundwater levels would rise by the amount shown on Figure 9.3-3 with an average EAA Reservoir A-1 depth of 8.8 feet. The seepage to STA-3/4 plus the seepage to the Supply Canal (which is assumed to be released to STA-3/4) is estimated to be approximately 10 percent (63,000 acre-feet per year) of the STA-3/4 treatment capacity of 600,000 acre-feet per year. The additional flow to the Holey Land is estimated to be approximately 3,600 acre-feet per year. Groundwater levels in STA-3/4 and the Holey Land would rise by up to two feet when the EAA Reservoir A-1 is at its average water depth of 8.8 feet, as shown in Figure 9.3-3. Figure 9.3-4 shows the increase in groundwater levels if the EAA Reservoir A-1 is maintained at its full depth of 12 feet for a long period of time.

9.3.3 Seepage Control Alternative 3

This alternative includes a 34-foot deep cutoff wall and a seepage canal with a bottom depth of 13.5 feet around the northwest, north, and east sides of the EAA Reservoir A-1. Also there is a 10-foot deep cutoff wall and no seepage canal along STA-3/4 and the Holey Land. In addition, there is an existing seepage canal along the north side of the Supply Canal, which is adjacent to the Holey Land. This seepage canal controls seepage into the farmland north of the Holey Land. The water level in the seepage canal is held 3.5 feet lower than the water level in the surrounding farmlands. The water collected by the seepage canal is pumped back into the EAA Reservoir A-1. Groundwater model results indicate that the total seepage from EAA Reservoir A-1 is approximately 269 cfs at an average EAA Reservoir A-1 depth of 8.8 feet based on the 35-year period of simulation from the water balance model. The model estimates that 249 cfs is collected by the seepage canal for this alternative. This alternative eliminates migration of seepage to the farmlands, causing a water level increase of less than 0.3 feet as shown in Figure 9.3-5 at an average EAA Reservoir A-1 depth of 8.8 feet. The seepage to STA-3/4 plus the seepage to the Supply Canal (which is assumed to be released to STA-3/4) is estimated to be approximately 10 percent (63,000 acre-feet per year) of the STA-3/4 treatment capacity of 600,000 acre-feet per year. The additional flow to the Holey Land is estimated to be approximately 3,600 acre-feet per year. Groundwater levels in STA-3/4 and the Holey Land would rise by up to two feet when the EAA Reservoir A-1 is at its average water depth of 8.8 feet, as shown in Figure 9.3-5. Figure

9.3-6 shows the increase in groundwater levels if the EAA Reservoir A-1 is maintained at its full depth of 12 feet for a long period of time.

9.3.4 Seepage Control Alternative 4

This alternative includes a 34-foot deep cutoff wall and a seepage canal with a bottom depth of 10 feet along the west, north, and east sides of the EAA Reservoir A-1. Along STA-3/4 and the Holey Land, there is a 10-foot deep cutoff wall and no seepage canal. In addition, there is an existing seepage canal along the north side of the Supply Canal, which is adjacent to the Holey Land. This seepage canal controls seepage into the farmland north of the Holey Land. This alternative includes a series of 550 pressure-relief wells spaced 100 feet apart along the seepage canal. The wells are drilled to a depth of 100 feet and have a diameter of 6 inches to capture deep seepage. The wells could be pumped in several ways. Alternative 4a includes linking together sets of approximately 25 wells by a header. Each set of 25 wells is connected to a pump station with a capacity of approximately 8.7 cfs that discharges to the seepage canal. This alternative includes a total of approximately 21 pump stations around the EAA Reservoir A-1. Alternative 4b includes separate pumps in each well, each with a capacity of approximately 150 gpm, which discharge to the seepage canal.

The total flow rate captured by the wells is 82,500 gpm (184 cfs) when the EAA Reservoir A-1 has 12 feet of water. At an average EAA Reservoir A-1 water depth of 8.8 feet, the total flow rate captured by the well is estimated to be 150 cfs. The water collected by the wells would be pumped into the seepage canal which would be maintained at the same level as the water levels in the surrounding farmlands. The seepage canal itself collects approximately 15 cfs of additional seepage from the EAA Reservoir A-1. This alternative essentially eliminates migration of seepage to the farmlands, causing a water level increase of less than 0.3 feet as shown in Figure 9.3-7. The seepage to STA-3/4 plus the seepage to the Supply Canal (which is assumed to be released to STA-3/4) is estimated to be approximately 10 percent (63,000 acre-feet per year) of the STA-3/4 treatment capacity of 600,000 acre-feet per year. The additional flow to the Holey Land is estimated to be approximately 3,600 acre-feet per year. Groundwater levels in STA-3/4 and the Holey Land would rise by up to two feet when the EAA Reservoir A-1 is at its average water depth of 8.8 feet, as shown in Figure 9.3-7. Figure 9.3-8 shows the increase in groundwater levels if the EAA Reservoir A-1 is maintained at its full depth of 12 feet for a long period of time.

9.3.5 Seepage Control Alternative 5

Similar to Alternative 4, this alternative includes a series of 275 pressure-relief wells spaced 200 feet apart. The wells are drilled to a depth of 100 feet and have a diameter of 6 inches to capture deep seepage. The wells could be pumped in several ways. Alternative 5a includes linking approximately 12 wells together by a header. Each set of 12 wells is connected to a pump station with a capacity of approximately 7.3 cfs that discharges to the seepage canal. This includes a total of approximately 23 pump stations around the EAA Reservoir A-1. Alternative 4b includes separate pumps in each well, each with a capacity of approximately 275 gpm, which discharge to the seepage canal.

The total flow rate captured by the wells is approximately 75,625 gpm (168 cfs) when the EAA Reservoir A-1 has 12 feet of water. At an average EAA Reservoir A-1 water depth of 8.8 feet, the total flow rate captured by the wells is estimated to be 150 cfs. The water collected by the wells would be pumped into the seepage canal which would be maintained at the same level as the water levels in the surrounding farmlands. The seepage canal itself collects approximately 15 cfs of seepage from the EAA Reservoir A-1. This alternative essentially eliminates migration of seepage to the farmlands, causing a water level increase of less than 0.3 feet, as shown on Figure 9.3-7. The seepage to STA-3/4 plus the seepage to the Supply Canal (which is assumed to be released to STA-3/4) is estimated to be approximately 10 percent (63,000 acre-feet per year) of the STA-3/4 treatment capacity of 600,000 acre-feet per year. The additional flow to the Holey Land is estimated to be approximately 3,600 acre-feet per year. Groundwater levels in STA-3/4 and the Holey Land would rise by up to two feet when the EAA Reservoir A-1 is at its average water depth of 8.8 feet, as shown in Figure 9.3-7. Figure 9.3-8 shows the increase in groundwater levels if the EAA Reservoir A-1 is maintained at its full depth of 12 feet for a long period of time.

Similar to Alternatives 4 and 5, another potential option is to connect the wells hydraulically to the seepage canal and, instead of pumping the wells directly, allow indirect extraction of deep seepage by lowering the seepage canal (i.e., a passive well system). Check valves would be required to prevent backflow of water in the seepage canal down the well. Calculations show that the seepage canal would need to be lowered by at least as much as for Alternative 2 in order to overcome well losses, friction in the well, and other minor head losses to induce groundwater flowback up the well. Capital cost for a passive well system would be more than for Alternative 2 and less than Alternatives 4 and 5. The idea of a passive well system may be considered during preliminary design.

9.4 COST, ADVANTAGES, AND DISADVANTAGES OF SEEPAGE CONTROL ALTERNATIVES

The advantages and disadvantages of each of the seepage control alternatives are given in Table 9.4-1 on the preceding page. Probable cost breakdowns are given in Table 9.4-2. Eliminating seepage impacts to STA-3/4 and the Holey Land will require extension of both the cutoff wall and seepage canal around the entire perimeter of the EAA Reservoir A-1, which would add significant cost to the Project. The other seepage control alternatives have a much lower cost but include the migration of seepage to some of the surrounding areas. The impacts to each of the areas are described below.

Table 9.4-1 Evaluation and Opinion of Probable Cost Comparison for Seepage Control Alternatives

Seepage Control Alternatives:	Alternative 1 **34-foot cutoff wall and 13.5-foot seepage canal around entire EAA Reservoir A-1, seepage canal held 3.5 feet below level in farmland**	Alternative 2 **34-foot cutoff wall and 10-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held at level in farmland**	Alternative 3 **34-foot cutoff wall and 13.5-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held 3.5 feet below level in farmland**	Alternative 4a **Pressure-relief wells spaced at 100 feet linked together in sets with a total of 21 pump stations of 3,900 gpm each; 34-foot cutoff wall and 10-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held at level in farmland[1]**	Alternative 4b **Pressure-relief wells spaced at 100 feet each with a 150 gpm pump; 34-foot cutoff wall and 10-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held at level in farmland[1]**	Alternative 5a **Pressure-relief wells spaced at 200 feet linked together in sets with a total of 23 pump stations of 3,300 gpm each; 34-foot cutoff wall and 10-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held at level in farmland[1]**	Alternative 5b **Pressure-relief wells spaced at 200 feet each with a 275 gpm pump; 34-foot cutoff wall and 10-foot seepage canal around west, north, and east sides; 10-foot cutoff wall and no seepage canal along STA-3/4 and Holey Land; seepage canal held at level in farmland[1]**
Location of drawing:	Figure 9.3-1	Figure 9.3-3	Figure 9.3-5	Figure 9.3-7	Figure 9.3-7	Figure 9.3-7	Figure 9.3-7
Relative advantages	Essentially eliminates seepage impacts to surrounding areas Prevention of seepage to STA-3/4 and the Supply Canal will maintain the capacity of STA-3/4 for treating surface water Pumping from the seepage canal involves less long term maintenance than alternatives including pressure-relief wells	Less pumping required from seepage canal than other alternatives Pumping from the seepage canal involves less long term maintenance than alternatives including pressure-relief wells	Seepage impacts are negligible in farmlands Pumping from the seepage canal involves less long term maintenance than alternatives including pressure-relief wells	Seepage impacts are negligible in farmlands Significant flexibility in controlling water levels at different locations around the EAA Reservoir A-1 Captures seepage near its exit point near the bottom of the cutoff wall	Seepage impacts are negligible in farmlands Significant flexibility in controlling water levels at different locations around the EAA Reservoir A-1 Captures seepage near its exit point near the bottom of the cutoff wall	Seepage impacts are negligible in farmlands Significant flexibility in controlling water levels at different locations around the EAA Reservoir A-1 Captures seepage near its exit point near the bottom of the cutoff wall	Seepage impacts are negligible in farmlands Significant flexibility in controlling water levels at different locations around the EAA Reservoir A-1 Captures seepage near its exit point near the bottom of the cutoff wall
Relative disadvantages	Must move embankment along STA-3/4 further to the north to provide space for seepage canal, which will reduce EAA Reservoir A-1 volume Significant seepage pumping is required Seepage canal must be deepened to allow for added drawdown Drawdown of seepage canal could dewater some areas near the canal at certain times of the year which may require monitoring and management of water levels in the seepage canal	Seepage migrates to farmlands Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water	Significant seepage pumping is required Seepage canal must be deepened to allow for added drawdown Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water Drawdown of seepage canal could dewater some areas near the canal at certain times of the year which may require monitoring and management of water levels in the seepage canal	Significant seepage pumping is required Future maintenance required for many wells and multiple pump stations Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water	Significant seepage pumping is required Future maintenance required for many pressure-relief wells and pumps Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water	Significant seepage pumping is required Future maintenance required for many pressure-relief wells and multiple pump stations Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water	Significant seepage pumping is required Future maintenance required for many pressure-relief wells and pumps Seepage to STA-3/4 and the Supply Canal will compete with ability to treat surface water
Opinion of Probable Cost for Seepage Control Over Life of Project							
Cost (present worth)	$ 229,000,000	$ 48,000,000	$ 65,000,000	$ 95,000,000	$ 92,000,000	$ 93,000,000	$ 80,000,000
Reference	Table 9.4-2	Table 9.4-2	Table 9.4-2	Table 9.4-2	Table 9.4-2	Table 9.4-2	Table 9.4-2

[1] Another potential alternative similar to Alternatives 4 and 5, with some similarities to Alternative 2, is a “passive” well system. The pressure-relief wells would be hydraulically connected to the seepage canal and the canal drawn down by over three feet to induce flow up the wells. This would eliminate individual well pumps and header pipes. The cost of this alternative would be higher than Alternative 2, and less than Alternatives 3 and 4. This concept remains a potential alternative for seepage control.

Table 9.4-2 Probable Cost for Seepage Control

Includes present worth cost over 50-year design life for features of the EAA Reservoir A-1 Project related to seepage control

Seepage Control Alternative	Added Embankment Cost for Seepage Control	PW Seepage Pumps Cost	Compensation to Farmers for Additional Pumping	Capital Costs of Vertical Wells and Pumps	PW Electrical Costs of Vertical Well Pumps	Total Cost Of Seepage Control Alternative
1	$119,018,000	$109,696,000	$0	$0	NA	$228,714,000
2	$0	$45,527,000	$2,473,000	$0	NA	$48,000,000
3	$0	$64,590,000	$0	$0	NA	$64,590,000
4a	$0	$42,530,000	$0	$46,847,000	$5,790,000	$95,167,000
4b	$0	$42,530,000	$0	$43,438,000	$5,834,000	$91,802,000
5a	$0	$42,530,000	$0	$45,172,000	$5,374,000	$93,076,000
5b	$0	$42,530,000	$0	$32,172,000	$5,344,000	$80,046,000

Description of Alternatives:

Alternative 1 – 34-foot deep cutoff wall and 13.5 foot deep seepage canal around entire EAA Reservoir A-1. Hold seepage canal water level 3.5 feet below water level in farms. Requires setback of embankment along Holey Land and STA-3/4.

Alternative 2 – 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level at same level as farms.

Alternative 3 - 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level 3.5 feet below water level in farms.

Alternative 4a - 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level at same level as farms. Install 550 pressure-relief wells along seepage canal, pumped at 150 gpm each. Header wells together and pump into seepage canal via 21 pump stations, each with a capacity of 3,900 gpm.

Alternative 4b - 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level at same level as farms. Install 550 pressure-relief wells along seepage canal. Each well has a 150 gpm pump that directly discharges to seepage canal.

Alternative 5a - 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level at same level as farms. Install 275 pressure-relief wells along seepage canal, pumped at 275 gpm each. Header wells together and pump into seepage canal via 23 pump stations, each with a capacity of 3,300 gpm.

Alternative 5b - 34-foot deep cutoff wall and seepage canal along northwest, north, and east sides of EAA Reservoir A-1. 10-foot deep cutoff wall and no seepage canal along Holey Land and STA-3/4. Hold seepage canal water level at same level as farms. Install 275 pressure-relief wells along seepage canal. Each well has a 275 gpm pump that directly discharges to seepage canal.

9.4.1 Farmlands

All alternatives except for Alternative 2 essentially eliminate seepage impacts to the surrounding farmlands. With Alternative 2, the network of farm canals, which are very effective at controlling water levels in the farmlands, would need to pump approximately 9,400 acre-feet per year of additional water as a result of the EAA Reservoir A-1 Project in order to maintain current water levels in the future. Alternatives 1 and 3 include maintaining the water level in the seepage canal at 3.5 feet lower than the water levels in the farmlands. This may cause some localized dewatering of the muck/peat in the vicinity of the seepage canal at certain times of the year depending on conditions at that time (weather, antecedent soil conditions, farm operations, etc.). It will be necessary to monitor and manage the degree to which the seepage canal is drawn down at these times. Alternatives 4 and 5 involving pressure-relief wells would provide added flexibility in controlling seepage in areas around the EAA Reservoir A-1 more than the other alternatives.

9.4.2 STA-3/4

Alternative 1 essentially eliminates the migration of seepage to STA-3/4, as shown on Figure 9.1-1, but the added cost to install additional depth of the cutoff wall and seepage canal around the entire EAA Reservoir A-1, plus the additional pumping required from the seepage canal is estimated to be between $134 and $181 M more than the other seepage control alternatives over the 50-year design life of the project. Alternatives 2 through 5 have nearly the same seepage impacts to STA-3/4 of approximately 10 percent of the 600,000 acre-feet per year treatment capacity of STA-3/4. This is approximately 1 percent of the 6,000 cfs designed hydraulic capacity of the STA-3/4. Seepage that flows through the aquifer into STA-3/4 could experience phosphorus removal since phosphorus may be adsorped and retained by soil particles. The impacts to STA-3/4 will include potential increases in water levels for Alternatives 2 through 5 (as shown on Figures 9.1-3, 9.1-5, and 9.1-7 at the end of the Section) at an average EAA Reservoir A-1 depth of 8.8 feet. These figures do not reflect proactive management of existing surface water control structures within STA-3/4 and the Supply Canal. Existing surface water structures might be used to reduce the amount of surface water that is released to STA-3/4 at times when seepage rates are high and possibly to distribute seepage more uniformly throughout STA-3/4 once the seepage rises to the surface.

9.4.3 Holey Land

Alternative 1 essentially eliminates the migration of seepage to the Holey Land, as shown on Figure 9.3-1, except for some seepage that will occur from the Supply Canal as the water level is increased to fill the EAA Reservoir A-1 for a portion of the year. Alternatives 2 through 5 have nearly the same seepage impacts to the Holey Land, including potential increases in water levels an average EAA Reservoir A-1 depth of 8.8 feet. (See Figures 9.1-3, 9.1-5, and 9.1-7 at the end of the Section) These figures do not reflect proactive management of existing surface water control structures within the Supply Canal or the Holey Land. Existing surface water structures might be used to reduce the amount of surface water that is released to the Holey Land at times when seepage rates are high and possibly to distribute seepage more uniformly throughout the Holey Land once the seepage rises to the surface.

9.4.4 U.S. 27

Because of the close proximity of U.S. 27 to the EAA Reservoir A-1, a two-dimensional SEEP/W groundwater model was developed to evaluate the potential influence of seepage on groundwater levels beneath the highway. The SEEP/W model allows more localized refinement of the model's aquifer characteristics in the vicinity of the highway than the large-scale MODFLOW model.

U.S. 27 is located along the east side of the EAA Reservoir A-1 Project site between the NNRC and the proposed seepage canal for EAA Reservoir A-1. The EAA Reservoir A-1 site is located approximately between Station 284+00 to the south and Sta. 700+00 to the north on the FDOT reference system for U.S. 27. The FDOT designs its roads and drainage systems to meet certain standards, and it is important that the EAA Reservoir A-1 Project does not compromise those standards.

Through meetings with FDOT, it has been established that two factors must be addressed by the design with respect to groundwater. Firstly, the water table must not rise into the road base material. The road base in this context was defined by FDOT as a layer approximately 18 inches beneath the road surface. Secondly, the water table must not be allowed to rise and diminish the storage volume within the drainage ditch system along the east side of the road which is used for environmental protection. SFWMD's Design Standard adjusted August, 2005 is that the groundwater level should be one foot below the bottom of a ditch during the dry season.

Along this section of road, FDOT drawings show that the lowest road surface elevation of the northbound and southbound lanes is approximately 15.6 feet NAVD88. This would place the bottom of the 18-inch base material at approximately 14.1 feet NAVD88, and the water table must be kept below this elevation beneath the road. The base of the drainage ditch, which runs along the west side of the road is approximately 11.6 feet NAVD88. Therefore, the water table must be kept below 10.6 feet NAVD88 during the dry season in order to keep the groundwater level one foot below this elevation along the east side of U.S. 27.

Simulations from the SEEP/W model were run for two cases: wet and dry seasons. The seepage canal was held at the same level as the water level in the surrounding farmlands, which is the same as Seepage Control Alternatives 2, 4, and 5 presented above. For Alternatives 1 and 3, the seepage canal is kept 3.5 feet below the levels in the surrounding farmlands. Therefore, the groundwater levels beneath the highway would be even lower than for the other alternatives and not as critical for the highway. The wet season was defined as those times when the water levels in the EAA Reservoir A-1, the seepage canal, the farm canals and water table are at their annual maximum. The dry season was defined as when the water levels in the seepage canal, the farm canals and water table are at their annual minimum but the water level in the NNRC is at its annual maximum due to agricultural practices.

The groundwater levels at shallow depth under the road and ditch are dominated by the NNRC and the seepage collection canal. The water table is virtually linear between the two canals. The results of the analyses are shown in Table 9.4-3. In the worst case, groundwater is 1.8 feet below the base of the ditch and 3.6 feet below the bottom of the 18-inch base material which is acceptable according to the criteria established by FDOT.

Table 9.4-3 Groundwater Elevations Beneath U.S. 27

(Elevations in feet NAVD88)

Seasonal Condition	**Elevation of Groundwater Beneath U.S. 27** (maximum permitted = 14.1 feet NAVD88)	**Elevation of Groundwater Beneath Drainage Ditch** (maximum permitted = 10.6 feet NAVD88)
Dry	10.5	9.8
Wet	9.0	8.5

9.4.5 Additional Seepage Evaluation

9.4.5.1 Wet Year Versus Average Year

Groundwater modeling was performed for October 1994 through September 1995, which was one of the wettest years on record with regard to antecedent moisture conditions and EAA Reservoir A-1 levels, as determined by the water balance model of EAA Reservoir A-1. For Alternative 2, the additional flow to the farmlands will increase by about 14 percent over a typical year, but for the other alternatives, the additional seepage would be controlled by additional pumping from the seepage canal or from pressure-relief wells. For all alternatives besides Alternative 1, the additional flow to STA-3/4 will increase by about 13 percent and the flow to the Supply Canal will increase by about 35 percent over a typical year. The additional flow to the Holey Land will increase by about eight percent over a typical year.

9.4.5.2 Incremental Effect of EAA Reservoir A-2

The USACE is evaluating the EAA Reservoir A-2 in a similar fashion as this evaluation of EAA Reservoir A-1, and their modeling has determined that there may be a need for additional cutoff wall depth along the south side of EAA Reservoirs A-1 and A-2 to limit the increase in water levels in the Holey Land. For all seepage control alternatives other than Alternative 1, the addition of EAA Reservoir A-2 and the incorporation of the same seepage controls for EAA Reservoir A-2 will cause more seepage to migrate to the Holey Land. For Alternative 1 (with a 34-foot cutoff wall all around the EAA Reservoir A-1), there is no significant increase in seepage to the Holey Land. To determine the potential magnitude of the incremental rise in water levels caused by adding EAA Reservoir A-2, a model run was performed for Seepage Control Alternative 3. With a shallow cutoff wall with a depth of 10 feet along the south side of the EAA Reservoirs A-1 and A-2, the incremental effect of adding EAA Reservoir A-2 is approximately 0.5 to 1.5 feet when the EAA Reservoir A-1 is held at a water depth of 8.8 feet. This increase is considered insignificant to the Holey Land.

9.4.5.3 Sensitivity Analyses

Many MODFLOW model runs were performed to determine the sensitivity of the model to the input parameters. Most notably, sensitivity analyses for the hydraulic conductivity and seepage canal conductance parameters were performed. Hydraulic conductivity values were determined from the Test Cell Program by calibrating the model to groundwater levels in many piezometers surrounding the Test Cells and to the measured seepage flow rates. If the calibrated vertical hydraulic conductivities of the aquifer layers are increased, there is more total seepage from the EAA Reservoir A-1, but more seepage is collected since it is easier for water to move upwards to the seepage canal. However, higher vertical hydraulic conductivities results in less impact to the surrounding areas. The sensitivity of the model to horizontal conductivities is greater than its sensitivity to vertical conductivities. By decreasing the horizontal conductivities, the total EAA Reservoir A-1 seepage decreases, and groundwater levels surrounding the EAA Reservoir A-1 decrease because there is more resistance to horizontal groundwater movement. It is believed, however, that calibration of hydraulic conductivities to the results of the Test Cell Program is the most appropriate and representative means of determining these parameters for the entire EAA Reservoir A-1 site.

The sensitivity of the model to the conductance of the seepage canal was also evaluated. The conductance is dependent on the width of the canal, the material at the bottom of the canal, and the hydraulic conductivity of the bottom of the canal (it determines the degree of communication between the canal and the aquifer). Since the seepage canal will be dug into the aquifer and will not initially have any sediment deposition, the conductance should be a high value (i.e., there is very little restriction to flow between the aquifer and the canal). Over time, some sediment may accumulate at the bottom of the seepage canal and provide some resistance to flow. Currently in the model, a conservative estimate of conductance is being used assuming one half foot for the thickness of the sediment. If the seepage canal conductance is increased, more seepage is collected by the seepage canal, and groundwater levels in the surrounding areas decrease. Currently, the model for EAA Reservoir A-1 and the USACE model of EAA Reservoirs assume the same conductance for the seepage canal.

9.5 MONITORING PROGRAM

A monitoring program of groundwater levels in the farmland should be initiated during the construction of the EAA Reservoir A-1 and continued after construction is completed. This will provide information to the SFWMD for evaluating the effectiveness of the seepage control. The monitoring program is a means to document whether flood (seepage) protection has been provided as required by the project assurances, which are discussed in Section 4.

9.6 SEEPAGE CONTROL SUMMARY

The most effective seepage control would be to line the entire bottom of EAA Reservoir A-1 with impermeable material or a synthetic liner, but the cost of doing so would be prohibitive. The next most effective seepage control would be to construct cutoff walls around the EAA Reservoir A-1 to a depth of 200 feet or more to a known layer of less permeable material. Again, the cost is prohibitive. Therefore, other more cost-effective seepage control alternatives were evaluated. Alternative 1, including a 34-foot cutoff wall and a 10 foot seepage canal surrounding the entire EAA Reservoir A-1 and maintaining the water level of the seepage canal below the level in the surrounding farmlands, would be the most effective of the five alternatives evaluated. Alternative 1 prevents migration of seepage to the farmlands, STA-3/4, and the Holey Land, and prevents impacts to U.S. 27. However, this alternative includes a significantly higher PW cost, between $134 and $181 M more than the other alternatives evaluated, mainly due to the additional cutoff wall depth and additional pumping over the life of the project.

The other seepage control alternatives allow migration of seepage to the Holey Land and STA-3/4, but essentially eliminate impacts to farms and U.S. 27. It is estimated that the total volume of seepage that would impact STA-3/4 and the Holey Land is on the order of one to 10 percent of the volume of water for which these areas were designed. Modeling results for Alternative 3 indicate that maintaining the water level of the seepage canal below the water levels in the farmlands is effective at preventing offsite migration of seepage, and this alternative includes the lowest present worth cost of the alternatives that prevent migration of seepage to the farmlands. The installation of pressure-relief wells as described by Alternatives 4 and 5, is predicated upon capturing deep seepage at the point where water passes beneath the bottom of the cutoff wall. The wells would be screened below the bottom of the cutoff wall, providing capture of deep seepage. Alternatives 4 and 5 include higher present worth costs for wells, pipes, and pumps than Alternative 3 of approximately $15.5 to $30.5 M over the 50-year design life of the project.

Alternatives 2 and 3 are the lowest cost alternatives. Alternative 3 allows the SFWMD more control of the pumping rates in the seepage canal than alternative 2, which relies on the farmers to pump the seepage. Alternative 3 is recommended.

Figure 9.3-1 Seepage Control Alternative 1 and Impact on Groundwater Levels at the Average EAA Reservoir A-1 Depth of 8.8 Feet

Configuration:

- EAA Reservoir A-1 at average depth of 8.8 feet
- 34-foot deep cutoff wall around entire EAA Reservoir A-1
- Seepage canal around entire EAA Reservoir A-1
- Seepage canal held 3.5 feet below farm water levels

Results:

- This alternative keeps rise in groundwater levels less than 0.3 feet in all of the areas surrounding the EAA Reservoir A-1 (farms, STA-3/4, and Holey Land) and keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 341 cfs (~246,900 acre-feet per year)
- Seepage canal collects 433 cfs (~313,500 acre-feet per year) from seepage and induced flow from surrounding canals
- Seepage control cost for Alternative 1 = $228,714,000

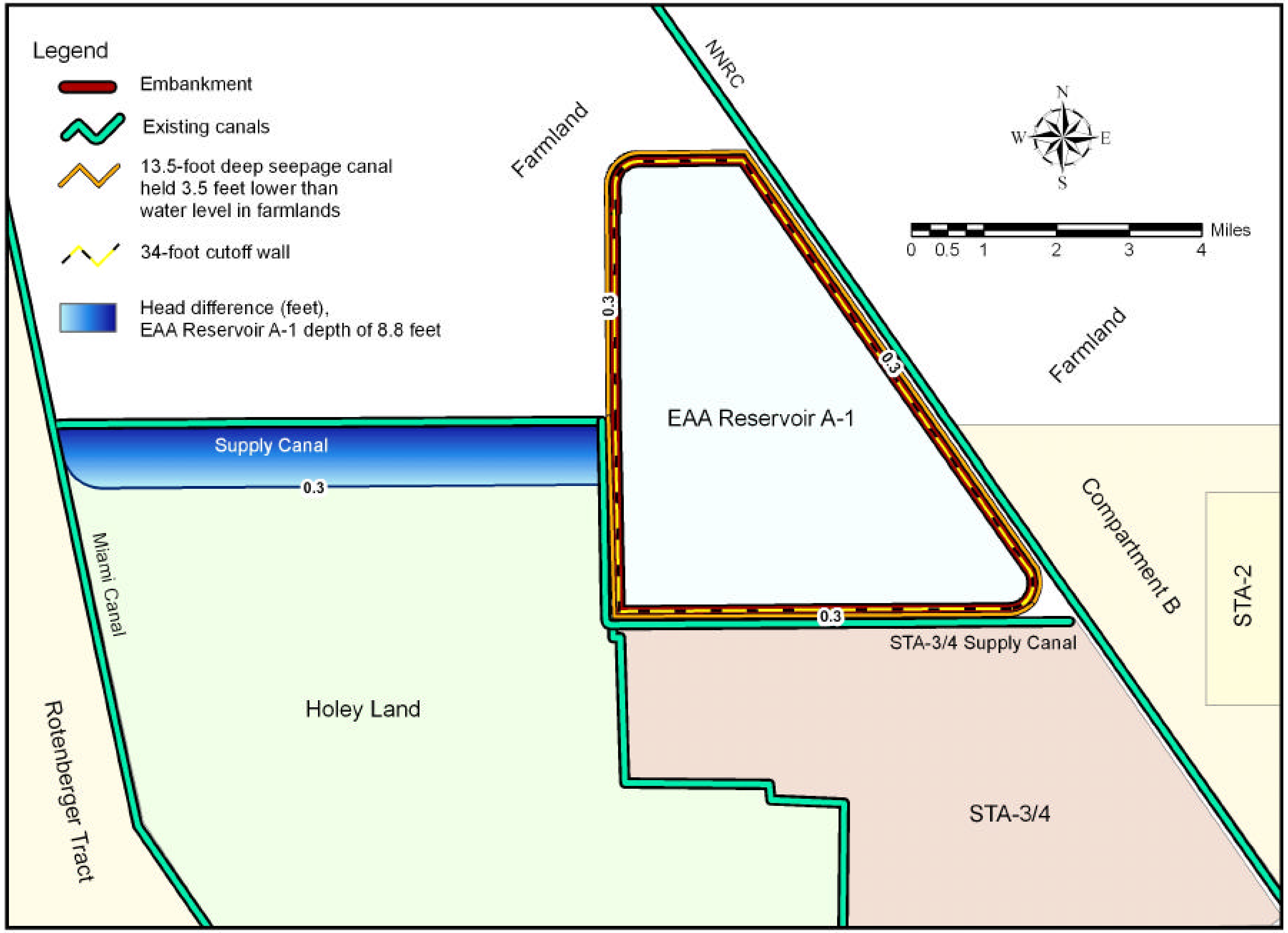


Figure 9.3-2 Seepage Control Alternative 1 and Impact on Groundwater Levels at the Full EAA Reservoir A-1 Depth of 12 feet

Configuration:

- EAA Reservoir A-1 at full depth of 12 feet
- 34-foot cutoff wall around entire EAA Reservoir A-1
- Seepage canal around entire EAA Reservoir A-1
- Seepage canal held 3.5 feet below farm water levels

Results:

- This alternative keeps rise in groundwater levels less than 0.3 feet in all of the areas surrounding the EAA Reservoir A-1 (farms, STA-3/4, and Holey Land) and keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 425 cfs (only when EAA Reservoir A-1 is full)
- Seepage canal collects 496 cfs from seepage and induced flow from surrounding canals.
- Seepage control cost for Alternative 1 = $228,714,000

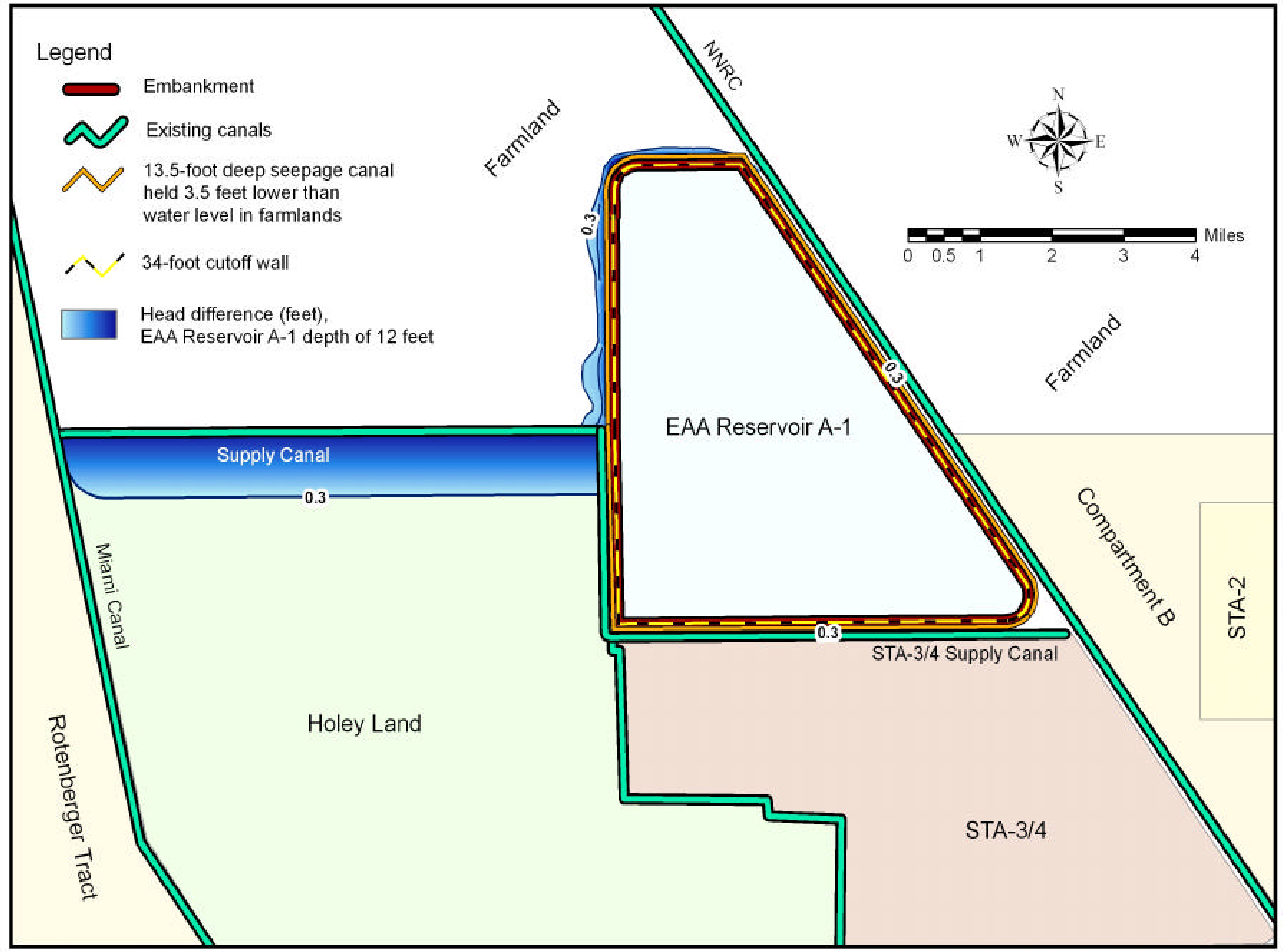


Figure 9.3-3 Seepage Control Alternative 2 and Impact on Groundwater Levels at the Average EAA Reservoir A-1 Depth of 8.8 feet

Configuration:

- EAA Reservoir A-1 at average depth of 8.8 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held at same level as farm water levels. Assumes farms, STA-3/4, and Holey Land are not operated to offset the rise in groundwater levels

Results:

- This alternative causes a rise in groundwater levels of up to 2 feet in the farms, STA-3/4, and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 230 cfs (~166,500 acre-feet per year)
- Seepage canal collects 174 cfs (~126,000 acre-feet per year)
- Total additional flow to farms is approximately 13 cfs (or approximately 9,400 acre-feet per year)
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet.
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 2 = $48,000,000

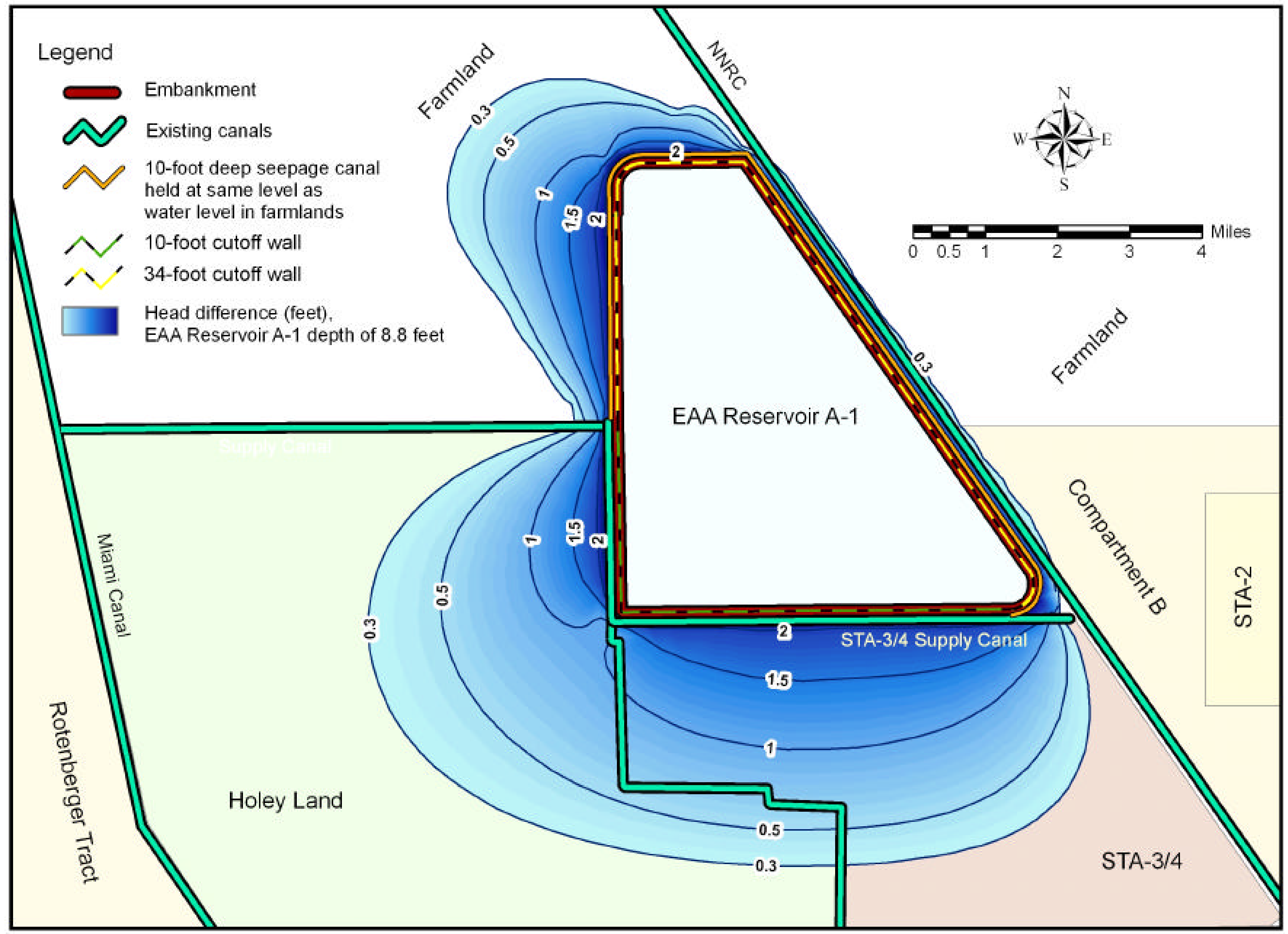


Figure 9.3-4 Seepage Control Alternative 2 and Impact on Groundwater Levels at the Full EAA Reservoir A-1 Depth of 12 feet

Configuration:

- EAA Reservoir A-1 at full depth of 12 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held at same level as farm water levels. Assumes farms, STA, and Holey Land are not operated to offset the rise in groundwater levels

Results:

- This alternative causes a rise in groundwater levels of up to 2.5 feet in the farms, STA-3/4, and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 304 cfs (only when EAA Reservoir A-1 is full)
- Seepage canal collects 212 cfs
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet.
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 2 = $48,000,000

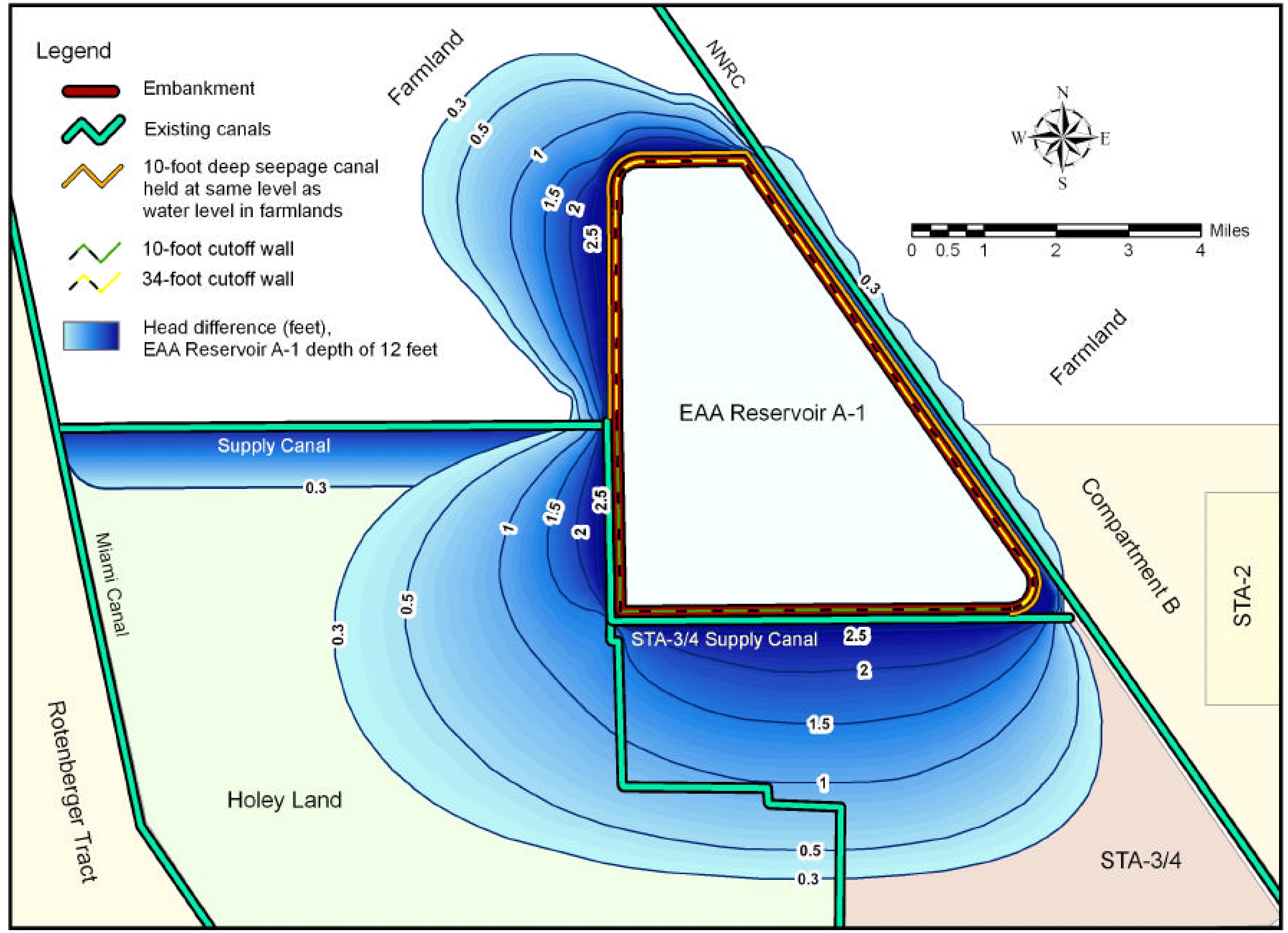


Figure 9.3-5 Seepage Control Alternative 3 and Impact on Groundwater Levels at the Average EAA Reservoir A-1 Depth of 8.8 Feet

Configuration:

- EAA Reservoir A-1 at average depth of 8.8 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held 3.5 feet below farm water levels. Assumes STA and Holey Land are not operated to offset the rise in groundwater levels

Results:

- This alternative keeps rise in groundwater levels in farms less than 0.3 feet
- This alternative causes a rise in groundwater levels of up to 2 feet in STA-3/4 and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 269 cfs (~194,700 acre-feet per year)
- Seepage canal collects 249 cfs (~180,300 acre-feet per year)
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 3 = $64,590,000

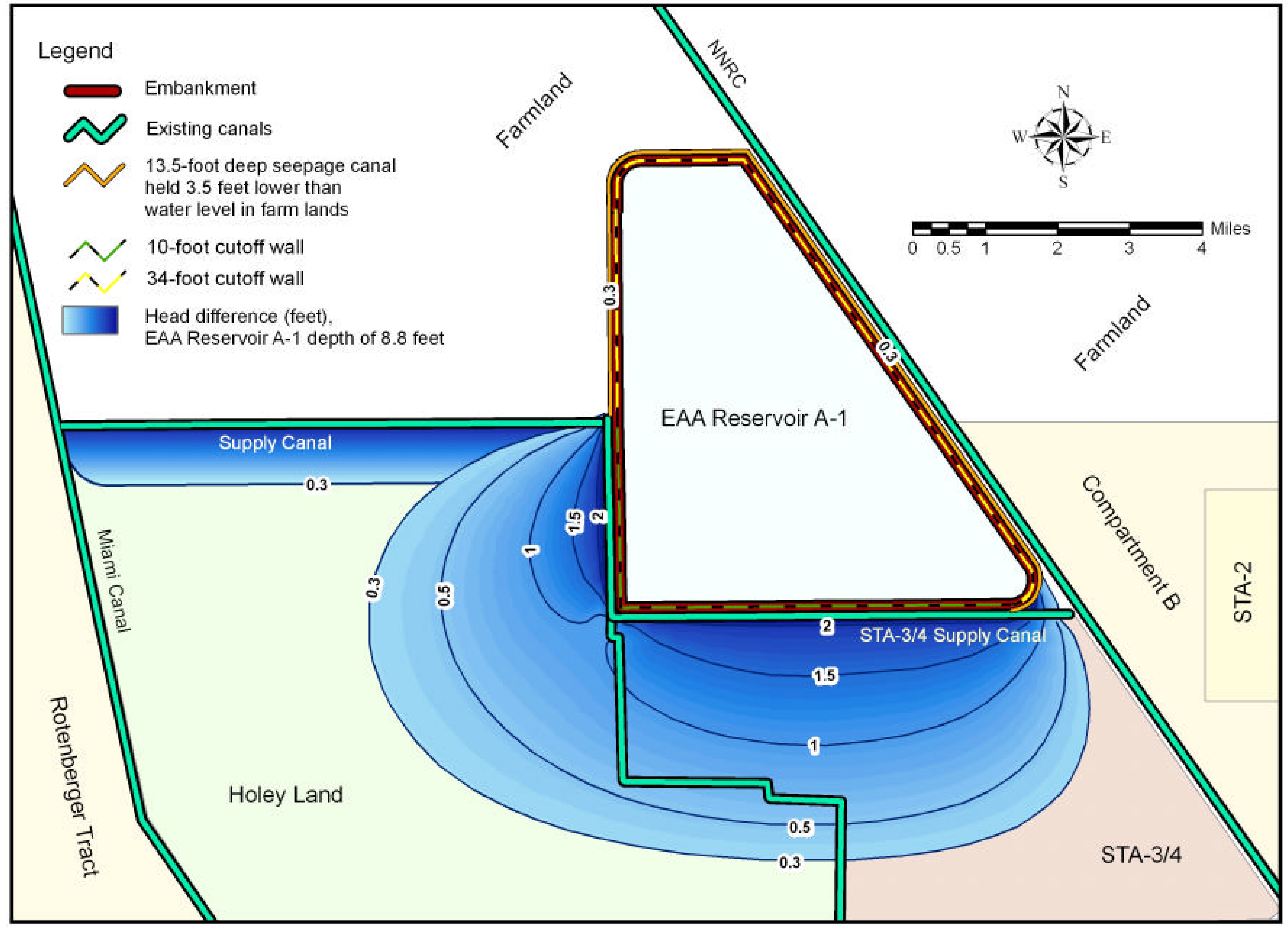


Figure 9.3-6 Seepage Control Alternative 3 and Impact on Groundwater Levels at the Full EAA Reservoir A-1 Depth of 12 feet

Configuration:

- EAA Reservoir A-1 at full depth of 12 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held 3.5 feet below farm water levels. Assumes STA and Holey Land are not operated to offset the rise in groundwater levels

Results:

- This alternative keeps rise in groundwater levels in farms less than 0.3 feet
- This alternative causes a rise in groundwater levels of up to 2.5 feet in STA-3/4 and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total EAA Reservoir A-1 seepage = 346 cfs (only when EAA Reservoir A-1 is full)
- Seepage canal collects 290 cfs
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet.
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 3 = $64,590,000

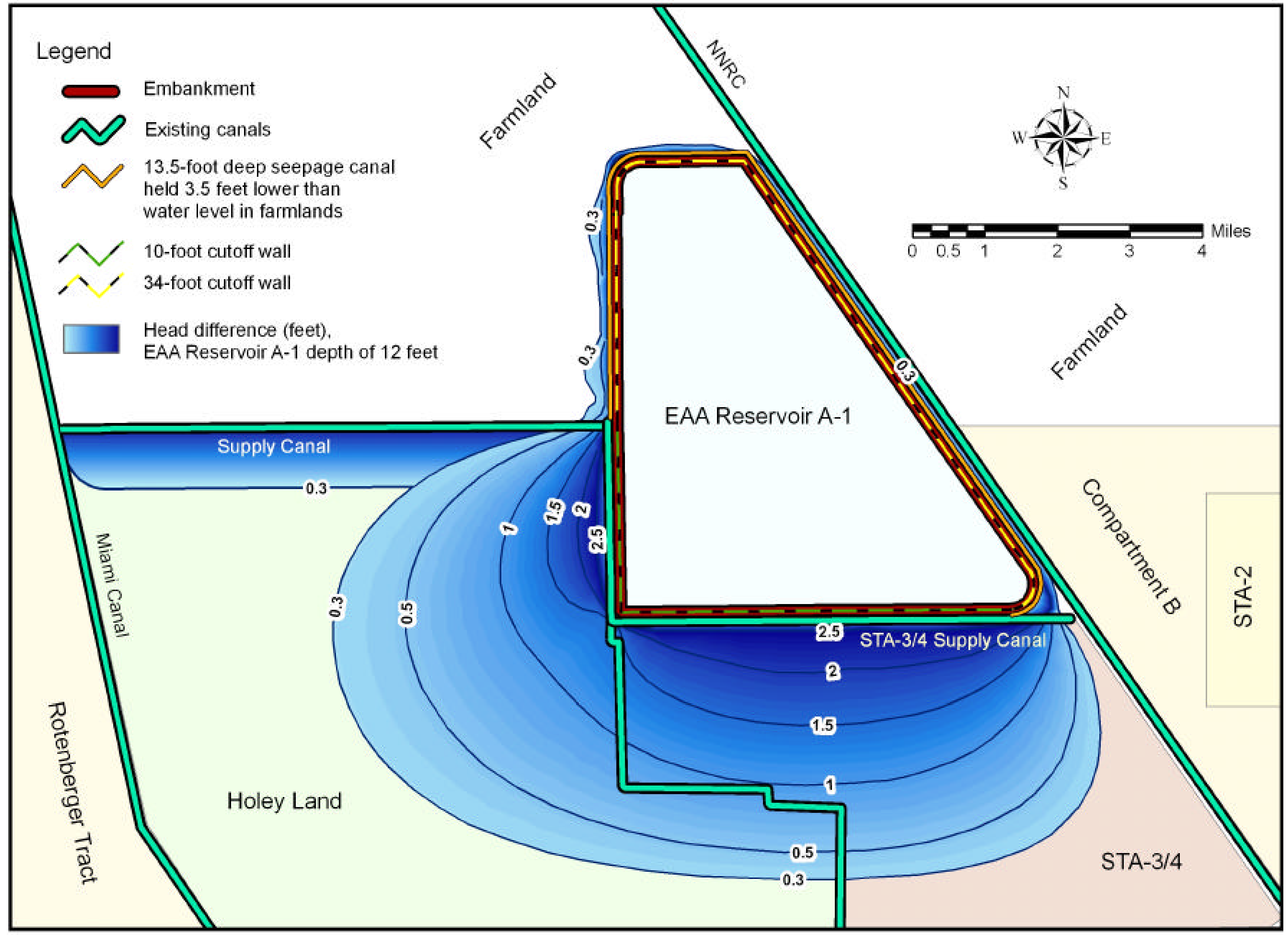


Figure 9.3-7 Seepage Control Alternatives 4 and 5 and Impact on Groundwater Levels at the Average EAA Reservoir A-1 Depth of 8.8 Feet

Configuration:

- EAA Reservoir A-1 at average depth of 8.8 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held at same level as farm water levels. Assumes STA and Holey Land are not operated to offset the rise in groundwater levels
- Alternative 4a - 550 wells drilled 100 feet deep, diameter 6 inches, spaced at 100 feet. Header wells together to 21 pump stations each with a capacity of 3,900 gpm
- Alternative 4b - 550 wells drilled 100 feet deep, diameter 6 inches, spaced at 100 feet. Each well has an individual 150 gpm pump.
- Alternative 5a - 275 wells drilled 100 feet deep, diameter 6 inches, spaced at 200 feet. Header wells together to 23 pump stations each with a capacity of 3,300 gpm
- Alternative 5b - 275 wells drilled 100 feet deep, diameter 6 inches, spaced at 200 feet. Each well has an individual 275 gpm pump

Results:

- These alternatives keep rise in groundwater levels in farms less than 0.3 feet
- These alternatives cause a rise in groundwater levels of up to 2 feet in STA-3/4 and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal.
- Wells and seepage canal collect an estimated 165 cfs (119,500 acre-feet per year) (for each alternative)
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 4a = $95,167,000
- Seepage control cost for Alternative 4b = $91,802,000
- Seepage control cost for Alternative 5a = $93,076,000
- Seepage control cost for Alternative 5b = $80,046,000

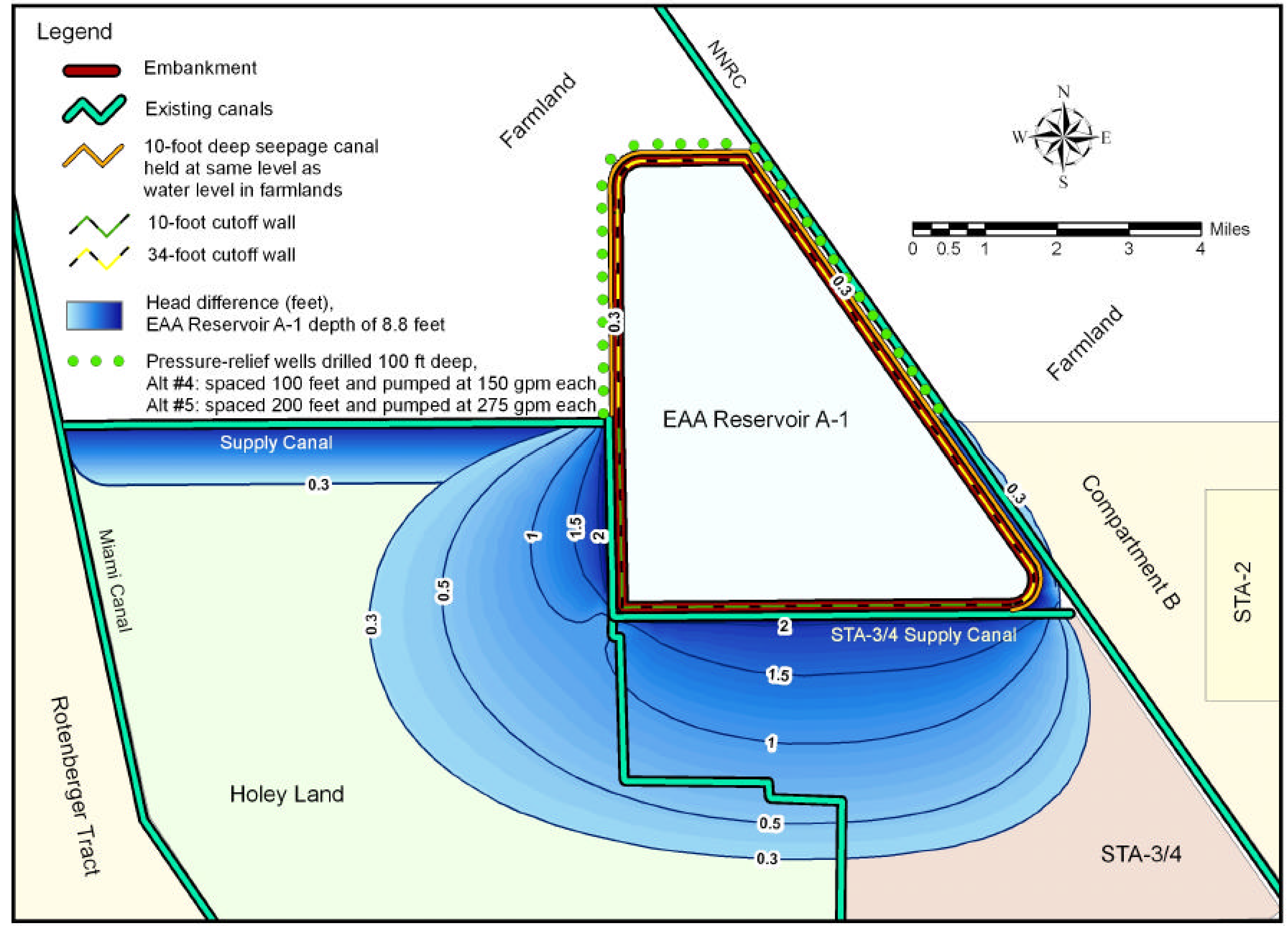


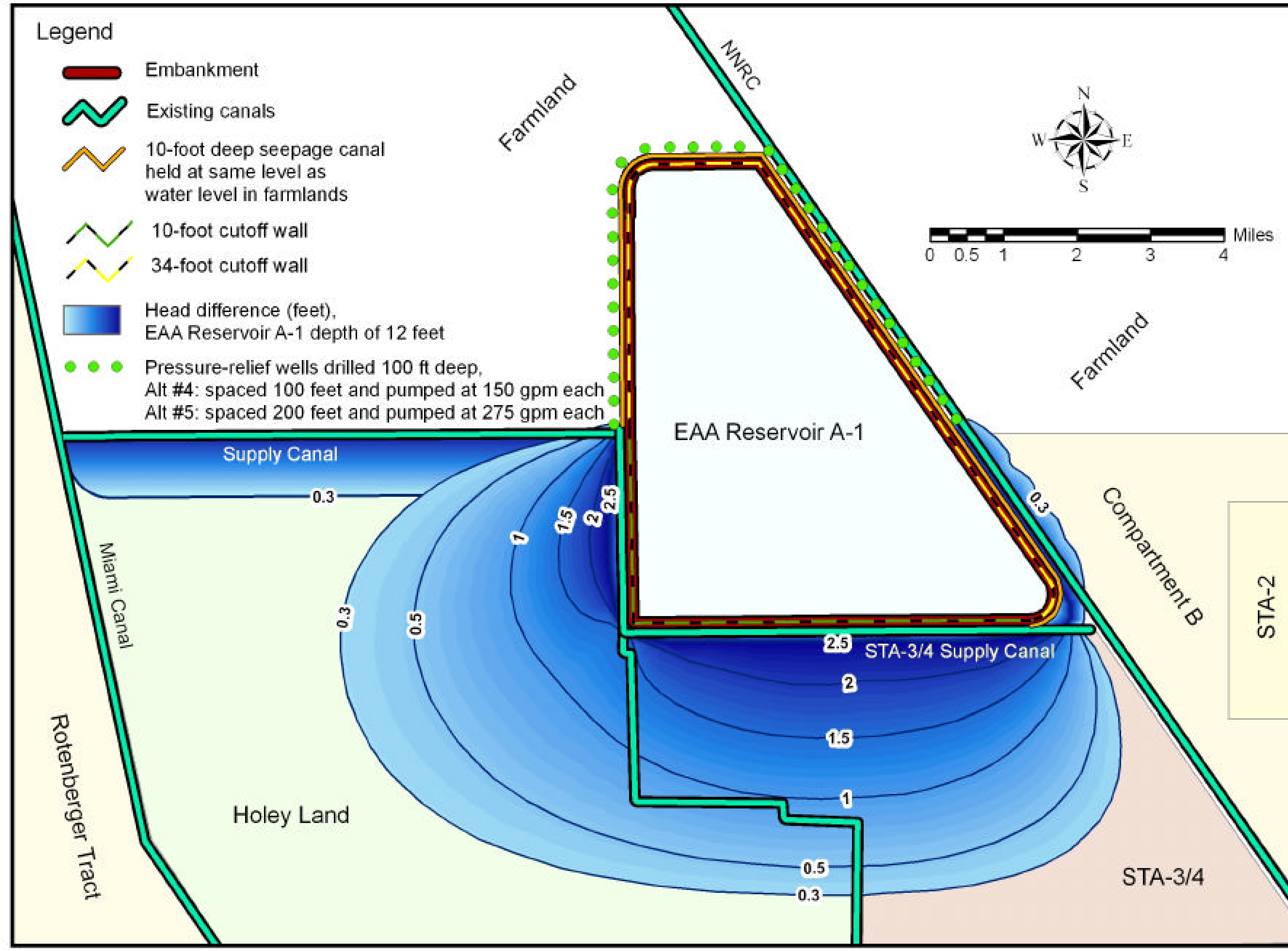
Figure 9.3-8 Seepage Control Alternatives 4 and 5 and Impact on Groundwater Levels at the Average EAA Reservoir A-1 Depth of 12 Feet

Configuration:

- EAA Reservoir A-1 at average depth of 8.8 feet
- 34-foot cutoff wall and seepage canal around northwest, north, and east sides of EAA Reservoir A-1; 10 foot cutoff and no seepage canal along STA-3/4 and Holey Land
- Seepage canal held at same level as farm water levels. Assumes STA and Holey Land are not operated to offset the rise in groundwater levels
- Alternative 4a - 550 wells drilled 100 feet deep, diameter 6 inches, spaced at 100 feet. Header wells together to 21 pump stations each with a capacity of 3,900 gpm
- Alternative 4b - 550 wells drilled 100 feet deep, diameter 6 inches, spaced at 100 feet. Each well has an individual 150 gpm pump.
- Alternative 5a - 275 wells drilled 100 feet deep, diameter 6 inches, spaced at 200 feet. Header wells together to 23 pump stations each with a capacity of 3,300 gpm
- Alternative 5b - 275 wells drilled 100 feet deep, diameter 6 inches, spaced at 200 feet. Each well has an individual 275 gpm pump

Results:

- At 12-foot EAA Reservoir A-1 depth a pumping rate from the wells and seepage canal of about 200 cfs
- These alternatives keep rise in groundwater levels in farms less than 0.3 feet
- These alternatives cause a rise in groundwater levels of up to 2.5 feet in STA-3/4 and Holey Land. Keeps groundwater at acceptable levels beneath U.S. 27. Some seepage will migrate to the Holey Land during periods of the year when the water level in the Supply Canal is elevated to fill the EAA Reservoir A-1. Seepage to the north of the Supply Canal will be controlled by an existing seepage canal
- Total additional volume to STA-3/4 is approximately 10,900 acre-feet per year with a maximum additional flow rate of approximately 29 cfs when Supply Canal is floated to 8 feet
- Net additional volume to the Supply Canal is approximately 30,100 acre-feet per year. Maximum flow into Supply Canal is approximately 84 cfs when EAA Reservoir A-1 is full of water. Maximum flow out of Supply Canal to surrounding areas is approximately 60 cfs when Supply Canal is floated to 8 feet.
- Total additional volume to Holey Land is approximately 9,900 acre-feet per year with a maximum additional flow rate of approximately 39 cfs when Supply Canal is floated to 8 feet
- Seepage control cost for Alternative 4a = $95,167,000
- Seepage control cost for Alternative 4b = $91,802,000
- Seepage control cost for Alternative 5a = $93,076,000
- Seepage control cost for Alternative 5b = $80,046,000



SECTION 10

CANALS

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TABLES

FIGURES

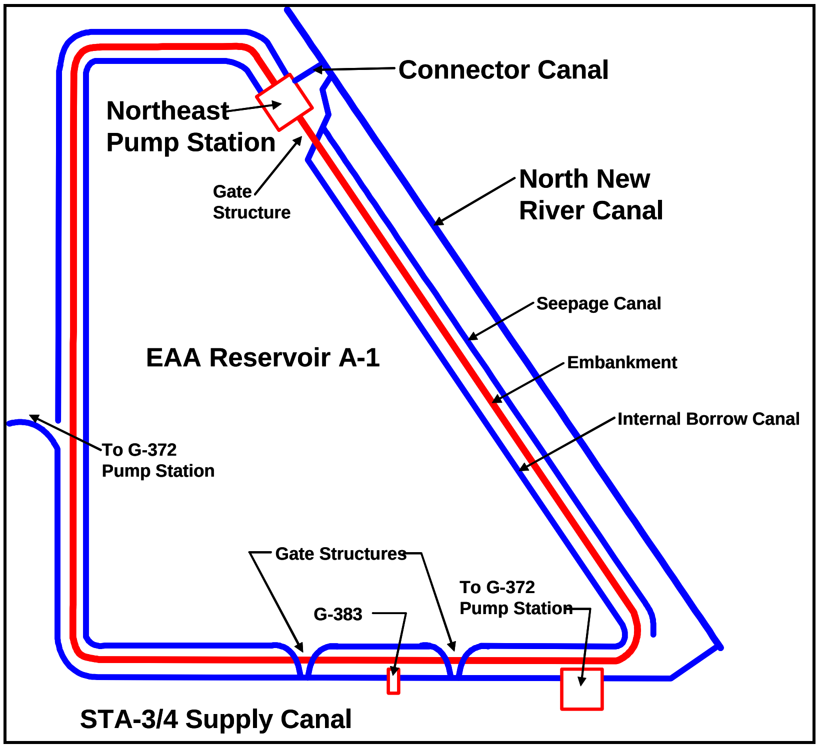
10. CANALS

10.1 INTRODUCTION

The EAA Reservoir A-1 Project is impacted by seven canal systems, most of which are shown in Figure 10.1-1. These are:

- North New River Canal (NNRC)
- Seepage collection canals
- Northeast pump station connector canal
- Internal perimeter borrow canal
- STA-3/4 Supply Canal
- Agricultural canals
- Miami Canal

Figure 10.1-1 Canals Served by EAA Reservoir A-1



10.2 NORTH NEW RIVER CANAL

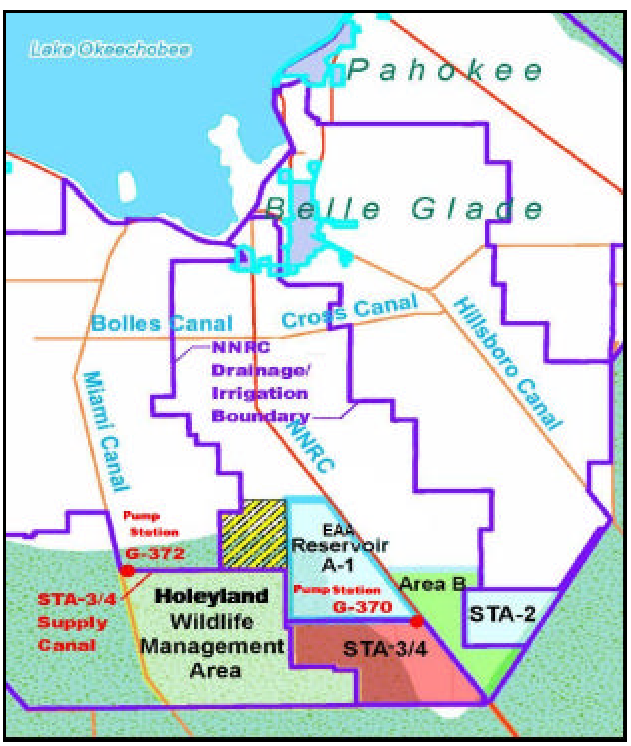
The NNRC extends from Lake Okeechobee south past the proposed EAA Reservoir A-1 as shown on Figure 10.2-1. The canal is intersected by the Bolles and Cross Canals north of the proposed EAA Reservoir A-1 site.

The NNRC is a bi-directional canal. During the wet season, the NNRC will convey local runoff from the adjacent farmlands and Lake Okeechobee releases south to the EAA Reservoir A-1. During the dry season, the water stored in EAA Reservoir A-1 can be conveyed north to meet agricultural deliveries.

The NNRC has the following characteristics:

- Earthen side slopes
- Irregular profile and cross-sections
- Several "choke" points between Lake Okeechobee and the G-370 pump station (located at the southeast corner of the EAA Reservoir A-1 site) including high points near the intersection with the Bolles and Cross Canals referred to as the "hump"
- Flow is directed to and from Lake Okeechobee, structure S-2, the Bolles and Cross Canals, and structure S-7
- An existing service area for drainage and irrigation of approximately 116,000 acres

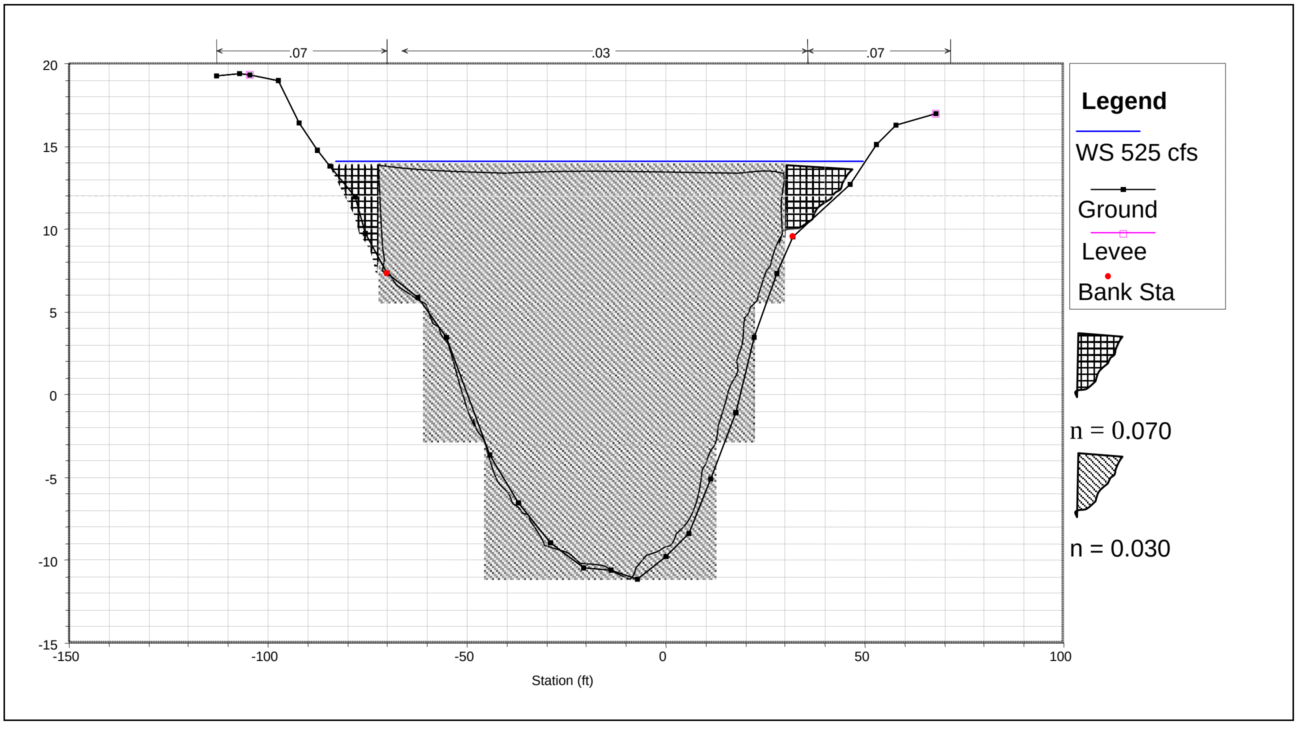
Figure 10.2-1 Location of the NNRC



10.2.1 Existing NNRC Characteristics

A HEC-RAS model containing data for the existing North New River, Miami, Bolles, and Cross Canals was obtained from the SFWMD. This model included cross-sectional data at various stations along each canal as well as Manning's "n" values for each cross-section. Figure 10.2-2 depicts a typical NNRC cross-section, and shows that the Manning's "n" value varies within the typical canal cross-section. The lower sections of the canal are relatively smooth and are assigned a Manning's "n" value of 0.030. However, the upper banks are likely to have vegetation and less smooth characteristics and are, therefore, assigned a Manning's "n" value of 0.070.

Figure 10.2-2 Typical NNRC Cross-Section



Detailed results and a summary of the selected model evaluations are provided in Appendix 6-4 Head Loss.

A.D.A. Engineering, Inc. has conducted the Regional Feasibility Study, which was initiated under the auspices of Florida's "Long-Term Plan for Achieving Everglades Water Quality Goals." The focus of the study as it pertains to the EAA Reservoir A-1 Project, is on re-distributing flows and phosphorus loads to optimize the water quality treatment function of the Everglades Construction Project stormwater treatment areas. The study was completed in October 2005. The amount of water available at the intersection of the NNRC and Cross Canals will be evaluated during later phases of the Regional Feasibility Study and further assessed as part of the next phase of design.

A 1953 USACE report indicates that maximum velocities have been determined as 3.0 feet per second (fps) in areas of sand and other unconsolidated materials, and 5.0 fps in areas of rock

(Reference 2). In general, canal velocities remain below 3 fps for most of the NNRC's length for the flow rates shown in Table 10.2-1. For the purposes of this analysis, areas where the velocity would exceed 2.5 fps were also identified. The modeling indicates that there are areas where the velocity would exceed 2.5 fps. The cross-sections referenced in this Section correspond to the cross-sections used in the HEC-RAS modeling of the NNRC.

10.2.2 Existing NNRC Capacities

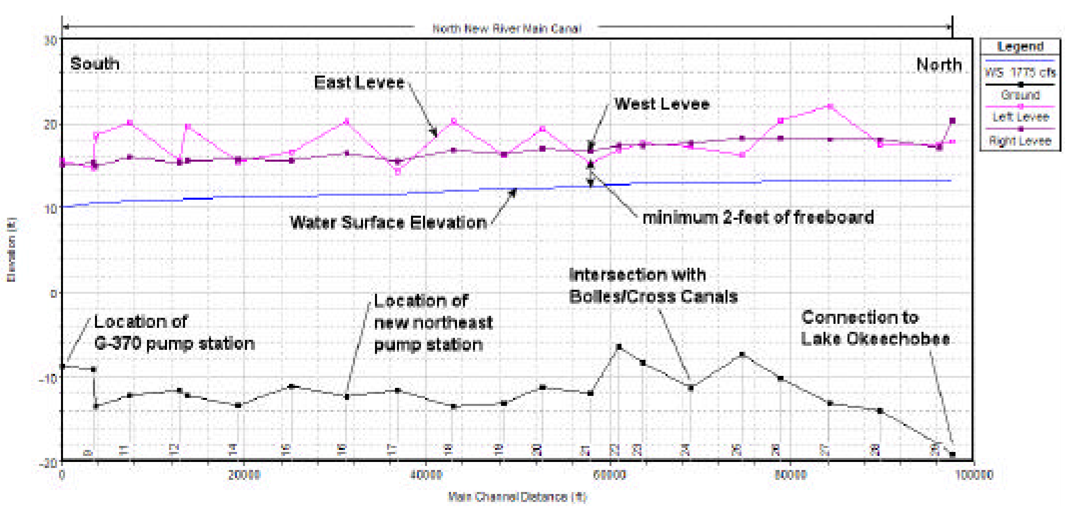
As part of Work Order 5, evaluations were performed to define the current NNRC hydraulic characteristics and capacities under a variety of operating conditions with respect to pumping from the G-370 pump station, pumping from the northeast pump station, lateral inflows from local precipitation events, and flows in and out of the NNRC from the Bolles and Cross Canals.

The hydraulic modeling of the NNRC to determine the available flow at the EAA Reservoir A-1 was based on the following assumptions:

- Flow rate available to flow south from the intersection of the Bolles and Cross Canals will be 2,000 to 3,000 cfs (Per ADA)
- Current average discharge during wet weather events is based on runoff of about 3/4-inch per acre per 24 hours
  - 1,015 cfs of lateral inflows between the Bolles and Cross Canals and the northeast pump station
  - 745 cfs of lateral inflows between the northeast pump station and G-370 pump station
- A minimum of two feet of freeboard is maintained at all points in the NNRC
- Water surface elevation at the intersection with the Bolles and Cross Canals cannot exceed 11.6 NAVD88

An example water surface profile from the NNRC hydraulic model runs is shown in Figure 10.2-3. Existing bottom-of-canal, east levee, and west levee ground surface profiles are also shown.

Figure 10.2-3 NNRC Current Conditions Profile (G-370 pump station to Lake Okeechobee)



The average wet weather discharge of 3/4-inch per acre per 24 hours is based on current permitted agricultural drainage pumping capacity. The permitted discharge from the agricultural areas is based on a formula that allows the largest runoff rates for the smaller farms with lesser rates as farms increase in acreage. The least amount of runoff allowed by formula is 3/4-inch per acre per 24 hours, and this applies to most of the NNRC drainage area. The maximum amount of runoff allowed is 1-1/2-inch per acre per 24 hours, but this applies only to a few of the smaller acreages. To put this into perspective, a rainfall event with an expected one year recurrence frequency is four inches per 24 hours. Therefore, rainfall events that produce more than 3/4-inch of runoff per 24 hours could be expected to occur multiple times each year.

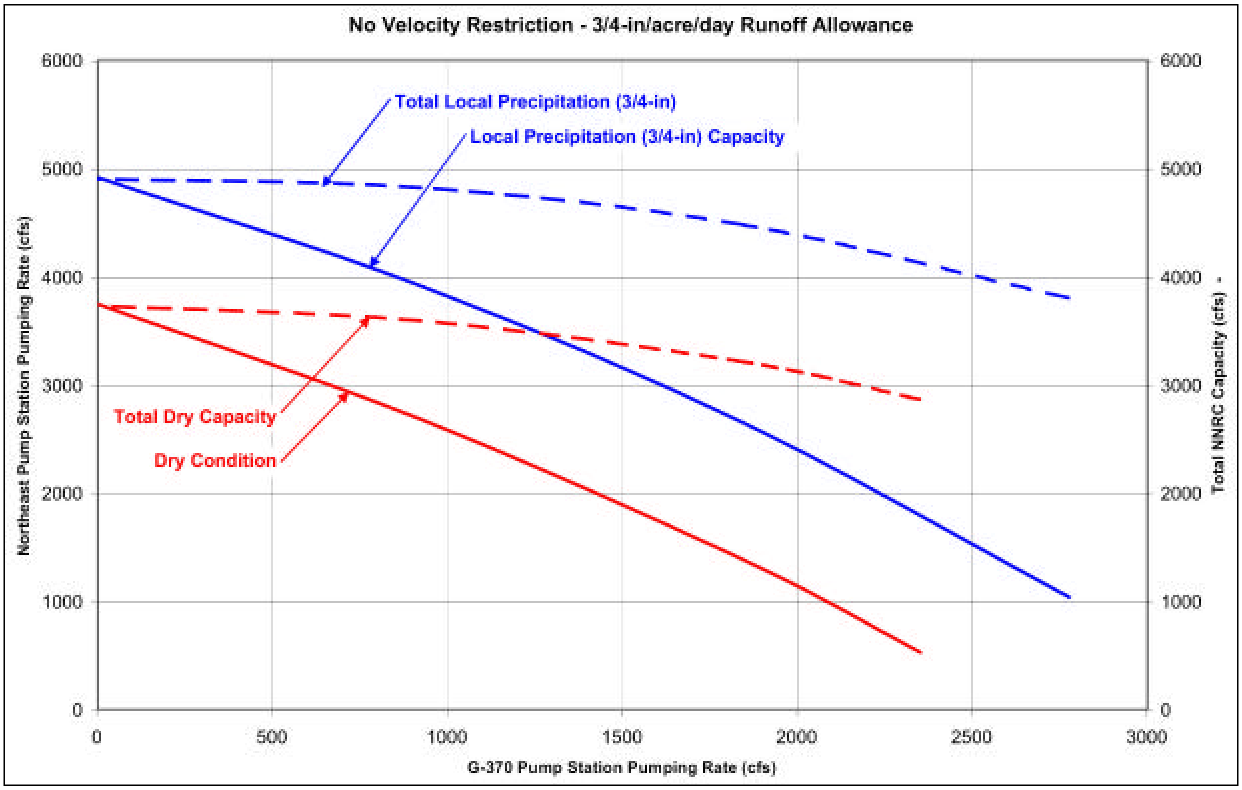
The location of the proposed northeast pump station relative to the Bolles and Cross Canals/NNRC intersection and to Lake Okeechobee results in a shorter distance of conveyance than that of the G-370 pump station as previously illustrated in Figure 10.2-1. Consequently, higher canal capacity can be achieved when a greater amount of flow is removed by the northeast pump station. This relationship is illustrated in Figure 10.2-4. In addition, local runoff from precipitation events will also result in shorter conveyance distances than would be experienced in the conveyance of dry weather discharges from Lake Okeechobee and subsequently higher conveyance capabilities will be realized. The difference in dry weather and wet weather capacity is also illustrated in Figure 10.2-4 and can be as much as 35 to 45 percent. Table 10.2-1 lists examples of multiple pump station configurations for both dry and local runoff of 3/4-inches in a 24-hour period. From the table, it is apparent that the total capacity diminishes as the capacity of the G-370 pump station increases, and total capacity increases as the capacity of the northeast pump station increases. Refer to Appendix 6-2 for additional discussion on existing canal capacities.

Table 10.2-1 NNRC Current Capacity with No Velocity Restrictions

Dry Conditions			Local Runoff		
G-370 Pump Station	**Northeast Pump Station**	**Total NNRC Capacity**	**G-370 Pump Station**	**Northeast Pump Station**	**Total NNRC Capacity**
0	3,740	3,740	0	4,910	4,910
925	2,675	3,600	925	3,905	4,830
1,850	1,370	3,220	1,850	2,635	4,485
2,350	525	2,875	2,350	1,800	4,150
2,775	-250[1]	2,775	2,775	1,035	3,810

[1] Based on analysis of data, NNRC does not currently have capacity to convey 2,775 cfs to G-370 Pump Station

Figure 10.2-4 NNRC Maximum Capacity with Respect to Pump Stations (No Maximum Velocity)



10.2.3 North New River Canal Modifications

Depending upon the selection of pump station capacities for the northeast pump station and G-370 pump station, canal modifications may or may not be necessary to the NNRC. As previously stated the NNRC is not currently sized to convey 2,775 cfs to G-370 pump station during dry weather conditions. Therefore, additional HEC-RAS model runs were performed to calculate the necessary cross-sectional area in the NNRC for various flow rates with 2,800 cfs always committed to G-370 Pump Station. The resulting cross-sections were then used to compute the amount of excavation required to meet the given capacity for the NNRC, and finally, a probable cost, including a 30% contingency was calculated. The results of this analysis are presented in Table 10.2-2. Excavation requirements are illustrated in Figure 10.2-5. In addition to cross-sectional modifications, it is important to note that the bottom-of-canal profile would be flattened to EL -14 NAVD88; this is shown in Figure 10.2-6 as the cross-hatched area. Finally, pump station capacity selection is discussed in detail in Section 6.5.

Figure 10.2-5 NNRC Typical Cross-Sectional Canal Modifications

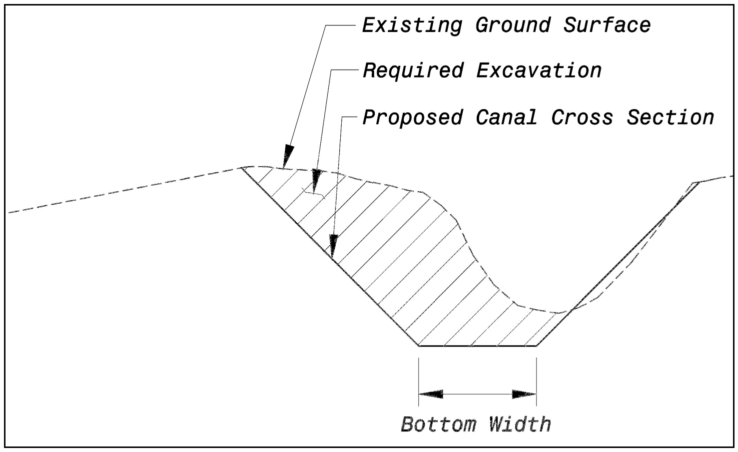


Figure 10.2-6 NNRC Bottom-of-Canal Profile Modifications

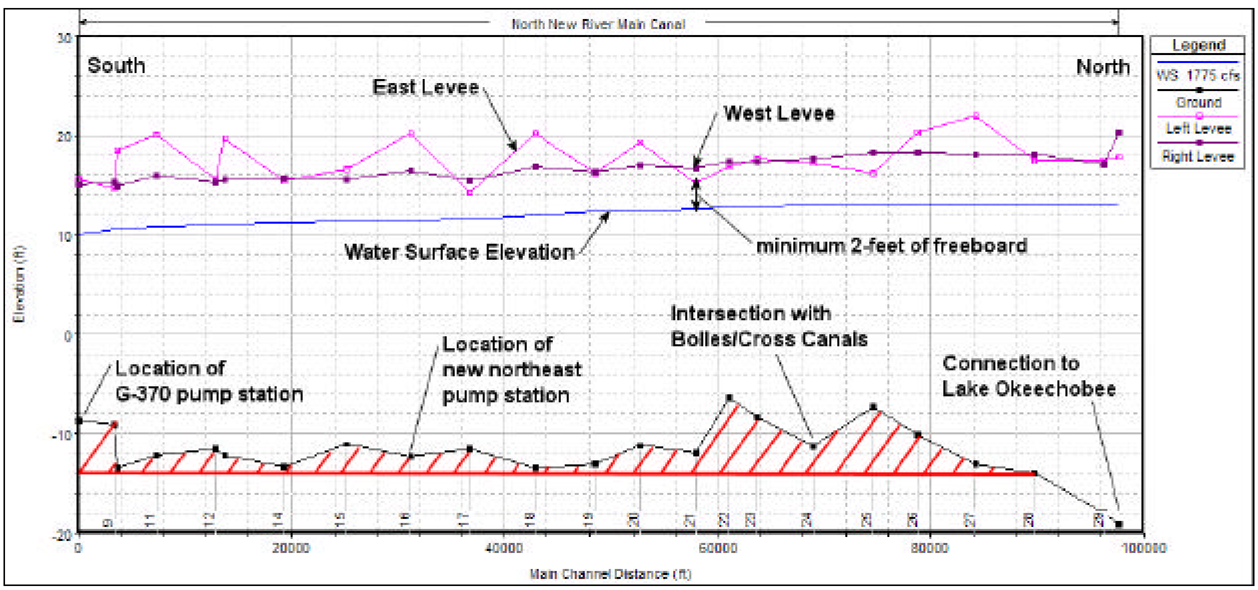
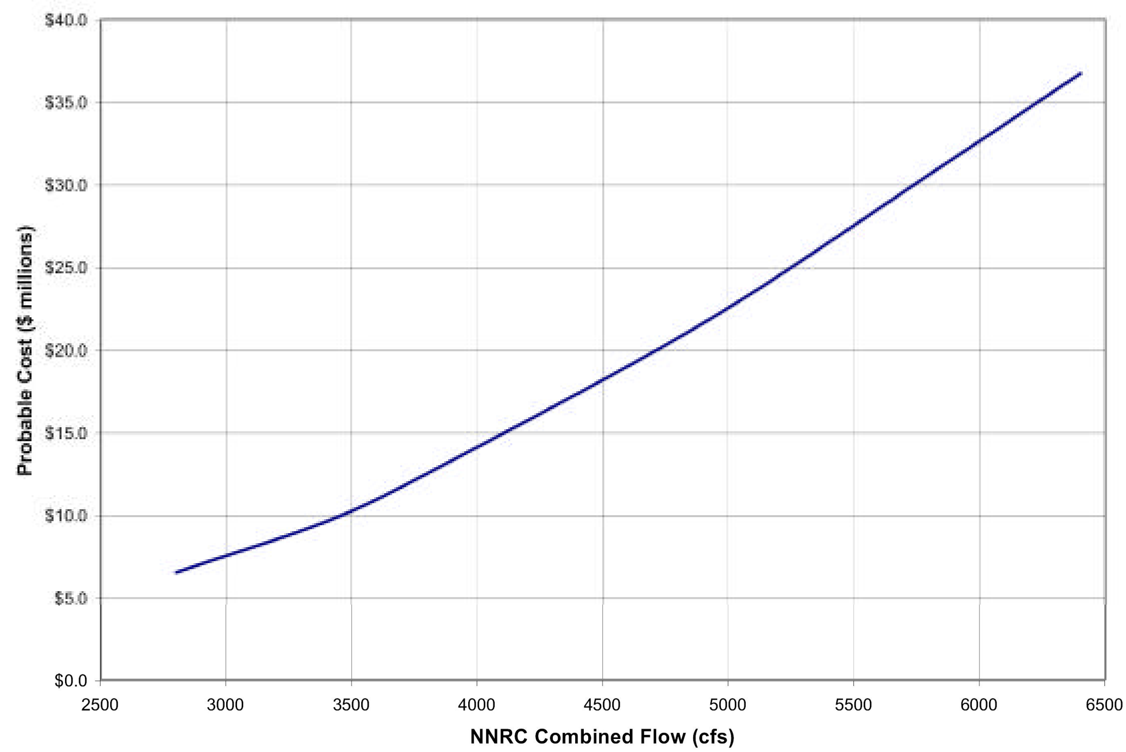


Table 10.2-2 NNRC Canal Modification Volumes and Costs

Combined Flow in NNRC (cfs)[1]	**Required Bottom Width at EL -14** (feet)	**Approximate Excavation Volume** (cubic yards)	**Probable Cost** ($)
2,800	11	617,730	$6,500,000
3,400	16	957,250	$9,598,000
3,800	20	1,279,280	$12,540,000
4,900	30	2,275,340	$21,560,000
5,800	39	3,275,360	$30,615,000
6,400	45	3,900,940	$36,719,000

[1] Flow capacity from Lake Okeechobee to the northeast pump station in cubic feet per second

Figure 10.2-7 Probable Cost of NNRC Modifications



10.3 SEEPAGE COLLECTION CANALS

The seepage canals around the exterior of EAA Reservoir A-1 will transport seepage from the EAA Reservoir A-1 to seepage pumps located at the northeast pump station. Additionally, the seepage collection canal will be connected to the existing seepage pumps at the G-370 pump station where those pumps will be utilized in emergency situations. The canal dimensions indicated below were determined based on seepage modeling performed with the intent of minimizing any increase in groundwater levels in the adjacent agricultural areas. A detailed discussion of this seepage modeling is presented in Section 9. Therefore, hydraulic calculations were based on the following assumptions for the seepage canal:

- 20-foot wide bottom, 2H:1V side slopes, and 13.5 foot average canal depth
- Unlined earthen side slopes with average Manning's "n" value of 0.030
- Minimum of two feet of freeboard and maximum of 3.5 feet of drawdown
- Maximum flow rate of 249 cfs (from seepage calculations as discussed in Section 9)
- Length of longest section of seepage canal is 41,000 feet
- For the Manning's equation calculation, the seepage canal is divided into 10 sections with equal seepage inflow of 13.6 cfs for average depth

Based on the hydraulic analysis of the seepage canals, the hydraulic headloss and velocities are not sufficient to cause erosion. Table 10.3-1 presents results (velocity and hydraulic drawdown) for various depths of water in the seepage canal.

Table 10.3-1 Velocity and Hydraulic Drawdown in the Seepage Canal

Average Water Depth (feet)	**Velocity at Pump Station** (feet per second)	**Drawdown at Pump Station** (feet)
8.0	0.47	0.16
7.5	0.52	0.21
7.0	0.57	0.27
6.5	0.63	0.36

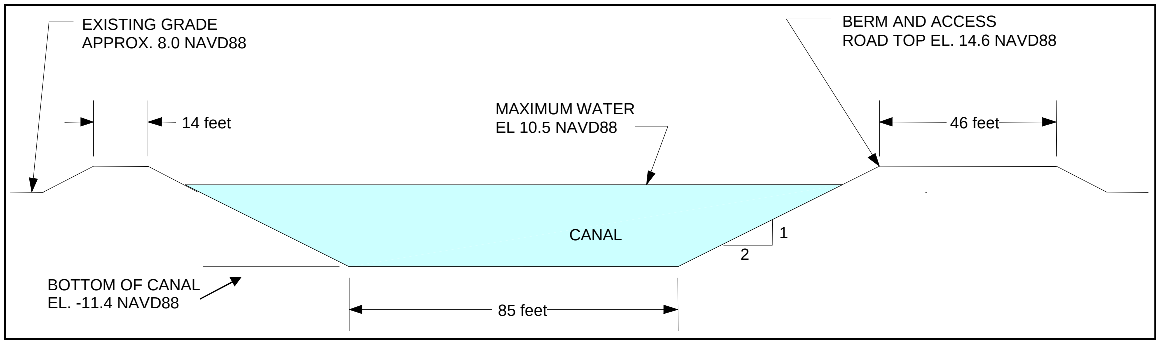
10.4 NORTHEAST PUMP STATION CONNECTOR CANAL

The connector canal will be constructed from the NNRC to the proposed northeast pump station. The following assumptions were made in order to determine an appropriate cross-section:

- Potential maximum northeast pump station and connector canal capacity of 3,600 cfs
- Potential maximum outflow from the EAA Reservoir A-1 is approximately 1,960 cfs, based on the agriculture deliveries provided by the water balance model
- A maximum length of canal from NNRC to pump station of 800 feet
- Side slopes of 2H:1V
- Maximum velocity of two feet per second
- Unlined earthen side slopes with an average Manning's "n" value of 0.030 for the entire perimeter
- A minimum water surface elevation in NNRC at pump station of 8.8 NAVD88, with elevation based on HEC-RAS modeling of NNRC
- A maximum water surface elevation in NNRC at pump station of 10.5 NAVD88, with elevation based on HEC-RAS modeling of NNRC
- Bottom of connector canal set at -11.4 NAVD88 (approximately two feet above the bottom of the NNRC at the northeast pump station

A hydraulic analysis of the connector canal indicated that a canal with a bottom width of 85 feet would produce a velocity of less than two fps and would result in a hydraulic drawdown of less than 0.03 feet from the NNRC to the northeast pump station. Figure 10.4-1 shows a typical cross-section through the connector canal. It will be necessary to construct berms above the existing grade on either side of the canal near the pump station, as the surrounding grade of approximately 8.0 NAVD88 is below the maximum canal water level of 11.5 NAVD88. The berms can also serve as a road for maintenance and access. The top of the berms will be at elevation 14.6 NAVD88. The design criteria require that all raised slopes are at 3H:1V for maintenance purposes, and that is sufficient to ensure stability of these low banks.

Figure 10.4-1 Typical Northeast Pump Station Connector Canal Cross-Section



10.5 INTERNAL PERIMETER CANAL

The primary function of this canal is to provide material for the zoned embankment option and the dimensions will be based accordingly. In the event that an RCC dam is constructed, there will be no need for this material and the internal perimeter canal would not be constructed. However, should an earthen embankment be constructed, sufficient material would be required and the internal canal would then extend around the entire perimeter of the EAA Reservoir A-1. The internal perimeter canal will interconnect with existing agricultural canals within the EAA Reservoir A-1, and therefore, improve drainage from the EAA Reservoir A-1 during low water levels. To some degree, the internal perimeter canals will address deep water refugia requested by FWC. The perimeter canal and existing agricultural canals would result in about three to five percent of the EAA Reservoir A-1 floor as deep water refugia.

10.6 STA-3/4 INFLOW SUPPLY CANAL

The STA-3/4 inflow Supply Canal currently conveys water from G-370 and G-372 pump stations to STA-3/4. The canal was designed to convey all flows pumped by the two pump stations which were sized to provide the design inflows to STA-3/4. No modifications in conveyance capacity are required.

10.7 AGRICULTURAL CANALS

The shallow agricultural canals immediately to the west of EAA Reservoir A-1 are currently drained or irrigated from the NNRC. This is illustrated in Figure 10.2-1 as the cross hatched area between the drainage area boundary and the EAA Reservoir A-1. A method will need to be developed to drain and irrigate this area after the EAA Reservoir A-1 is constructed and before the EAA Reservoir A-2 is constructed. Options that could address irrigation and drainage of this area include:

- Realignment of the drainage boundary and such physical modifications to provide drainage/irrigation service from the Miami Canal
- Modification to convey drainage/irrigation water to and from the STA-3/4 Supply Canal. Gates and pumps would be required

- Modifications to convey drainage/irrigation water via the EAA Reservoir A-1 seepage canal. If the seepage canal is allowed to be connected to the NNRC, it could be used to provide irrigation to, and drain water from, the agricultural area

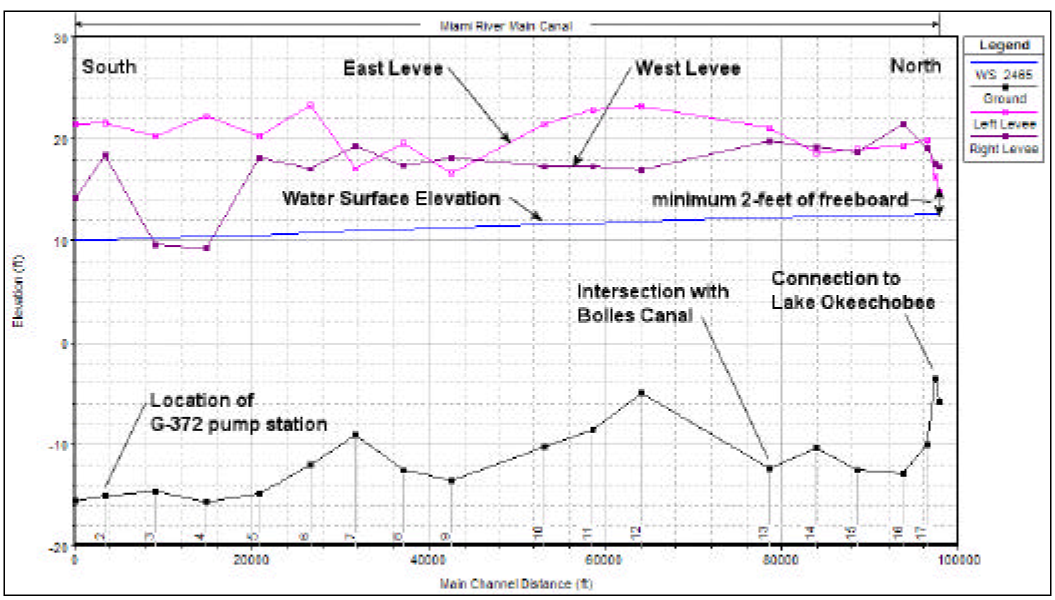
The method used to supply the agricultural canals will be finalized during the preliminary design.

10.8 MIAMI CANAL

The Miami Canal currently conveys water from Lake Okeechobee to the south. The exception to this would be during extreme wet weather conditions; excess runoff stored in the Miami Canal can be pumped into Lake Okeechobee through structure S-3. Flow from the Miami Canal is currently pumped by the G-372 pump station into the STA-3/4 Supply Canal to be treated by STA-3/4.

Information for the existing Miami Canal was obtained from the SFWMD's HEC-RAS model as previously stated. Analysis was performed using HEC-RAS to calculate the maximum capacity of the existing Miami Canal while maintaining a minimum of two feet of freeboard. Based on this data, the maximum capacity of the Miami Canal is 2,465 cfs. It was noted that two data points for the west levee of the Miami Canal directly north of the G-372 pump station were inconsistent with the surrounding elevations and were therefore regarded as corrupt data and ignored. The water surface profile resulting from this hydraulic modeling is presented in Figure 10.8-1.

Figure 10.8-1 Miami Canal Current Conditions Profile (G-372 pump station to Lake Okeechobee)



10.9 SLOPE STABILITY

Preliminary design of below ground canal slopes has been based largely on empirical evidence of existing structures and specific experience from Test Cell construction. Ground conditions comprise caprock over Fort Thompson Formation silts, sands and gravels. Below ground, the canal side slopes are 2H:1V.

Internal erosion or piping of fine sands and silts of the Fort Thompson Formation is a potential concern where water emerges from the ground into the new perimeter seepage collection canals. This concern is avoided by limiting the hydraulic gradient at this point using the embankment configuration, cutoff wall beneath the embankment, and distance of the canal from the embankment. A minimum factor of safety of three shall be maintained against this condition.

10.10 SEEPAGE CONTROL

In the EAA, water conveyance canals are frequently flanked by seepage collection canals to collect and control seepage. The EAA Reservoir A-1 Project provides new connections with existing water conveyance canals and will increase capacity of existing canals where necessary, but no new canals are included in the Project outside of the EAA Reservoir A-1. In general, existing seepage control measures will be retained adjacent to canals.

The one exception is on the south side of the EAA Reservoir A-1 where the STA-3/4 Supply Canal's northern levee would be incorporated into a new embankment and the northern seepage collection canal would be lost. If the EAA Reservoir A-1 water level is greater than the STA-3/4 Supply Canal water level, seepage would be from the EAA Reservoir A-1 into the Supply Canal and the STA-3/4. If the EAA Reservoir A-1 water level is lower than the STA-3/4 Supply Canal water level, seepage water will be collected within the EAA Reservoir A-1 and eventually returned to the system. This is an acceptable means of seepage control for the Supply Canal.

10.11 EROSION

Flow conditions have been assessed in the NNRC and northeast pump station connector canal. With no modifications, the maximum velocities in the NNRC remain below three fps. Modifications have been proposed that would reduce maximum velocity to less that 2.5 fps and the northeast pump station connector canal would also be designed to this standard. These velocities are sufficiently small to avoid erosion. In addition, should SFWMD opt for increased capacity in the NNRC, the modifications would be made in a manner to ensure low velocities to minimize erosion potential.

In general, it is not expected that erosion protection will be needed for the canal slopes. Early promotion of grass root development and periodic maintenance of canal slopes should result in stable conditions. Protection might be required in specific areas local to structures where velocities might be higher or where geometry might cause a flow concentration.

10.12 REFERENCES

HEC-RAS Canal Models, compiled as part of the South Florida Water Management District (SFWMD) *"Bolles & Cross Canals Preliminary-Hydraulics Report"*, South Florida Water Management District.

U.S. Army Corps of Engineers, Office of the District Engineer. *Partial Definite Project Report – Central and Southern Florida Project – For Flood Control and Other Purposes – Part I – Agricultural and Conservation Areas – Supplement 13,* Design Memorandum, Hydrology and Hydraulic Design of NNRC and Related Works (L-18, L-19, L-20, and S-7), Jacksonville, FLA., July 6, 1953, p 10.

SECTION 11

STRUCTURAL DESIGN CRITERIA

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TABLE

11. STRUCTURAL DESIGN CRITERIA

This section describes the basis of structural design for new or modified facilities.

11.1 APPLICABLE CODES AND STANDARDS

Design of structural elements will comply with the design codes and standards included in the Codes and Standards portion of Section 3.

11.2 DESIGN STRESSES

11.2.1 Minimum Concrete Compressive Strength (Unconfined)

• Mass concrete, concrete deduction factor (f'c)	3,000 pounds per square inch (psi) at 28 days
• Structural concrete, f'c	4,000 psi at 28 days

11.2.2 Reinforcing Steel

• ASTM A615, steel yield strength (fy)	60,000 psi

11.2.3 Structural Steel

• Wide flange shapes, ASTM A572, Grade 50, fy	50,000 psi
• Angles, channels and plates, ASTM A36, fy	36,000 psi
• Pipe sections, ASTM A53, Type E, fy	35,000 psi
• Tube sections, ASTM A500, Type B or C, fy	46,000 psi

11.2.4 Masonry

• Concrete masonry units (CMU), Grade N-1, compressive strength	1,900 psi
• Compressive strength of mortar, Type S	1,800 psi
• Compressive strength of grout	2,000 psi
• Masonry unit assembly, compression strength (f'm)	1,500 psi

11.3 LOADING CRITERIA

11.3.1 Dead Loads

• Equipment	Actual

• Phantom load	1 kip[1] at secondary beams 2 kips at primary beams
• Bridge crane or monorail	Actual crane beam + rail only
• Roof, superimposed	Actual, 15 pounds per square foot (psf) minimum

11.3.2 Live Loads, per Major Pumping Station Engineering Guidelines

• Roof (minimum, unreduced)	50 psf
• Operating floors	250 psf, or the heaviest piece of machinery anticipated to be placed therein, whichever is larger
• Control rooms	100 psf
• Restrooms	100 psf
• Equipment and storage rooms	200 psf
• Maintenance work areas	300 psf
• Stairways	100 psf
• Elevator lift and handicap ramp	200 psf
• Deck grating	250 psf
• Service bridge	HS-25 or SFWMD 40T truck crane loading (P&H 440TC), whichever is larger
• Guardrails (at top rail)	50 pounds per linear foot (plf) + 200 pound concentrated load, acting in any direction
• Bridge crane or monorail, vertical load	Rated capacity (full wheel load) + 25 percent impact
• Bridge crane or monorail, lateral and longitudinal loads	Lateral load = 20 percent of the sum of the weights of the lifted load + the crane trolley. Longitudinal load = 10 percent of the maximum wheel load.

[1] A unit of weight equal to 1,000 pounds or 455 kilograms

For large equipment areas the combined weight of equipment and base plus an additional live load of 50 psf over the base area will be used as the live load.

11.3.3 Lateral Loads

• Active earth pressure	Conducted at 30 percent design
• At-rest earth pressure	Conducted at 30 percent design
• Passive earth pressure	Conducted at 30 percent design
• Lateral surcharge load from compaction (decreases linearly)	400 psf at the ground surface, 0 psf at the depth equal to 400 psf divided by the earth pressure
• Hydrostatic	63 pounds per cubic foot (pcf)
• Vertical surcharge, at locations subject to truck or equipment loads	Surcharge shall be calculated based on the equipment listed in Section 11.3.2, subject to a 500 psf minimum

The active pressure values will only be used for site retaining walls that are free to rotate.

11.3.4 Snow Loads - Not Applicable

11.3.5 Seismic Loads

Earthquake loads will not be considered, in accordance with the 2004 Florida Building Code.

11.3.6 Wind Loads – Pump Station

• Design 3-second gust wind speed	155 mph
• Height and exposure coefficient	Exposure C
• Structure importance factor	1.51
• Building type	Partially enclosed

11.3.7 Wind Loads – Flood Control Elements

• Design 3-second gust - wind speed	130 mph
• Height and exposure coefficient	Exposure C
• Structure importance factor	1.30
• Building type	Partially enclosed

11.3.8 Flood Load (Hydrostatic + Wave)

• Dynamic Pressure Coefficient (ASCE 7-02, Table 5-2)	3.5

11.4 HYDRAULIC STRUCTURES AND PUMPING STATION SUBSTRUCTURE

11.4.1 Materials of Construction

Hydraulic structures and the new northeast pump station substructure will be constructed of reinforced concrete. Because water in the Everglades is aggressive to concrete, Type II Cement will be specified. Any platforming associated with these items will be constructed of aluminum shapes, aluminum grating and aluminum guardrail. Connection bolts will be either stainless steel or aluminum. Reinforced concrete platforming will be used in locations where the use of grating is not appropriate.

11.4.2 Design Procedures and Assumptions

Hydraulic structures and the new northeast pump station substructure will be designed based upon the loads, load combinations and allowable stresses contained in EM 1110-2-2104, subject to meeting the requirements of the SFWMD's Design Standards of August, 2005 and ACI 318-02. Temperature and shrinkage reinforcement and cracking limits will be in accordance with ACI 350.

- For reinforcement in shear, the required strength is 1.3 times the excess applied shear (Vu) less shear carried by the concrete (NVc). Thus NVs >1.3 (Vu-NVc), where NVs is the design capacity of shear reinforcement.
- Rectangular walls may be analyzed as two way rectangular plates when the aspect ratio of length to height is 2H:1V or less. The boundary conditions will be chosen to give reasonably conservative results. If the aspect ratio exceeds 2H:1V, the wall will be designed as a one way rectangular plate and the corners will be investigated assuming a 2H:1V ratio.
- The design of water containment walls will consider both flexure and tension in the walls. The horizontal reinforcement on the water side will be apportioned for 100 percent flexure steel plus 100 percent tension steel.
- Direct tension in the foundation and top slabs due to internal water pressure will be accounted for in the design of the slab's horizontal reinforcing. The foundation's top reinforcement will be assumed to resist 100 percent of the tension at the foundation. The tension in the top slab may be resisted equally between the top and bottom reinforcement for reasonably thin slabs.
- A minimum reinforcement for shrinkage and temperature will be provided in accordance with ACI 350. As indicated in ACI 350, a minimum reinforcement ratio of 0.5 percent will be provided in basin walls and base slab with a basin dimension of 50 feet or more in any direction. Reinforcement ratios in the direction where the structure dimension are less than 50 feet will be in accordance with ACI 350. Minimum size of shrinkage and temperature reinforcement will be #4 and will be divided equally between the two surfaces of the concrete section. Concrete sections greater than 24 inches thick may have minimum reinforcing

based on a 24-inch thickness. The shrinkage and temperature reinforcement in the bottom of slabs reinforced top and bottom, in contact with the subgrade, can be reduced to one-half the values calculated.

- Hydrostatic groundwater pressure for structures adjacent to the EAA Reservoir A-1 will be based on the water level of the EAA Reservoir A-1. In accordance with USACE EM 1110-2-2104, the uplift pressure distribution along the base of foundations will be assumed to be linear between the upstream and downstream edges of the foundation. The pressure distribution will be modified to take into account any foundation drains or groundwater cutoff devices. Uplift reduction at drains may not exceed 50 percent of the difference between the full uplift head at the pump station intake and the drain.

11.4.3 Design Load Cases

Listed below is a summary of the loading assumptions and load factors for design:

Where:

DL = Dead load

LL = Live load

Hw = Lateral hydrostatic pressure

Hweq = Lateral hydrodynamic pressure

Hs = Lateral Static Soil Load (including at-rest soil plus groundwater hydrostatic pressure, surcharge, and compaction pressures)

Fa = Flood load

W = Wind load

U = Required strength to resist factored loads

11.4.4 Service Water Condition

For a maximum service water level, ignore the soil backfill loads unless soil loads are additive to the overall loading on a structural element, and consider internal tensile forces in the wall with a hydraulic factor of 1.65, and load combinations as follows:

Flexure: U = 1.3[1.4(DL) + 1.7(LL) + 1.7(Hw)]

Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hw)]

Flexure and Shear: U = 0.9D + 1.6W + 1.6Hw

11.4.5 Flood/Overflow Water Condition

For maximum water level at the flood/overflow elevation (highest water elevation that could occur hydraulically, which is not necessarily at the top of wall), where the cracking limit is not applicable, ignore the soil backfill loads unless the soil loads are additive to the overall loadings on a structural element. Then consider internal tensile forces in the wall, and load combinations are as follows:

Flexure and Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hw)]

U = 1.2DL + 0.8 W + 1.0Fa + L + 1.7Hw

U = 0.9DL + 0.8 W + 1.0Fa + 1.6Hw

11.4.6 Seismic Water Condition

Earthquake loads will not be considered, in accordance with the 2004 Florida Building Code.

11.4.7 Service Soil Condition

For maximum soil backfill height with at rest pressure, with and without internal liquid loads, and the groundwater table is at its normal elevation, include soil compaction or soil surcharge whichever controls the load. Load combinations are as follows:

Flexure: U = 1.3[1.4(DL) + 1.7(LL) + 1.7(Hs)]+

Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hs)]

Flexure and Shear: U = 1.2DL + 0.8W + 1.0Fa + L + 1.7Hw

U = 0.9DL + 0.8W + 1.0Fa + 1.6Hw

11.4.8 Flood Soil Condition

For maximum soil backfill height with at-rest pressure plus hydrostatic pressure of groundwater at 100-year flood level and EAA Reservoir A-1 at maximum full storage level, with internal liquid loads, including soil compaction or soil surcharge whichever controls the load, load combinations are as follows:

Flexure and Shear: U = 0.75[1.4(DL) + 1.7(LL) + 1.7(Hs)]

U = 1.2DL + 0.8 W + 1.0Fa + L + 1.7Hw

U = 0.9DL + 0.8 W + 1.0Fa + 1.6Hw

Note: that the one-third allowable stress increase is included in the above equations.

11.4.9 Steel Hydraulic Structures

Steel hydraulic structures will be designed in accordance with the Allowable Stress Design Method listed in EM-1110-2-2105 and the AISC Manual of Steel Construction. Allowable stresses will be reduced by 0.83 in accordance with Type B modifications listed in Section 4-4 of EM-1110-2-2105. No corrosion allowance will be added to steel cross-sections.

11.4.10 Overturning, Sliding and Flotation

Overturning stability, sliding safety factor and the flotation safety factor shall be in accordance with the following values, based on service level loads, and neglecting live loads. (See Table 11.4-1)

Table 11.4-1 Overturning, Sliding, and Flotation Factors

Aspect	Usual	Unusual	Extreme
Percent of Base in Compression	100	75	Resultant must be within the base
Sliding Safety Factor	2	2	1.33
Flotation Safety Factor	1.5	1.3	1.1

11.5 BUILDING STRUCTURES

Building structures, excluding structural concrete, will be designed based upon the loads, load combinations and allowable stresses contained in the 2004 Florida Building Code. Structural concrete design will be based on strength design in accordance with the SFWMD Design Standard of August, 2005 and ACI 318-02. The additional concrete design requirements of ACI 350-01 and EM1110-2-2104 will not be considered applicable for building structures unless exposed to water, wastewater or aggressive chemicals such as saltwater. Additionally, building structures and their components that are subject to equipment impact and vibration will be designed in accordance with the applicable recommendations of ACI 350.4R subject to engineering judgment.

Lateral wind loads will be transferred to the foundation from their origin in a rational manner. The horizontal distribution of wind loads will be based upon the assumption that the roof/floor diaphragms are both rigid and flexible for steel deck diaphragms, and rigid for cast in place or precast concrete diaphragms. Where the diaphragm is assumed to behave in a flexible manner, the wind lateral load distribution will be based upon the tributary area to the resisting elements. Where the diaphragm is assumed to behave as a rigid panel, the wind lateral load distribution is based on the relative rigidities of the resisting elements.

11.6 INSPECTION REQUIREMENTS

Inspection will be required per the 2004 Florida Building Code, Chapters 1 and 17.

11.7 BRIDGE OVER CONNECTOR CANAL

A new bridge will be constructed to carry traffic on U.S. 27 over the new connector canal between the NNRC and the new northeast pump station. U.S. 27 is a divided highway at the planned location of the new connector canal, so the bridge will be a dual structure consisting of a reinforced concrete slab superstructure, supported on two end bents and intermediate bents. Each of the end and intermediate bents will consist of square prestressed concrete piles with reinforced concrete cap beams.

The bridge configuration under any conditions should maintain a minimum of two feet of freeboard above the design high water level of the connector canal.

The Bridge Analysis Report and Location Hydraulic Report for the bridge development process have not been completed. Completion of these reports will be made upon approval of the connector canal size and design, and the final location of the new northeast pump station has been established. The bridge will be designed in accordance with AASHTO, Standard Specifications for Highway Bridges, 17th Edition – 2000. The bridge will be designed for an HS25 loading.

SECTION 12

SITE CIVIL DESIGN

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FIGURES

12. SITE CIVIL DESIGN

12.1 EAA RESERVOIR A-1 CONFIGURATION

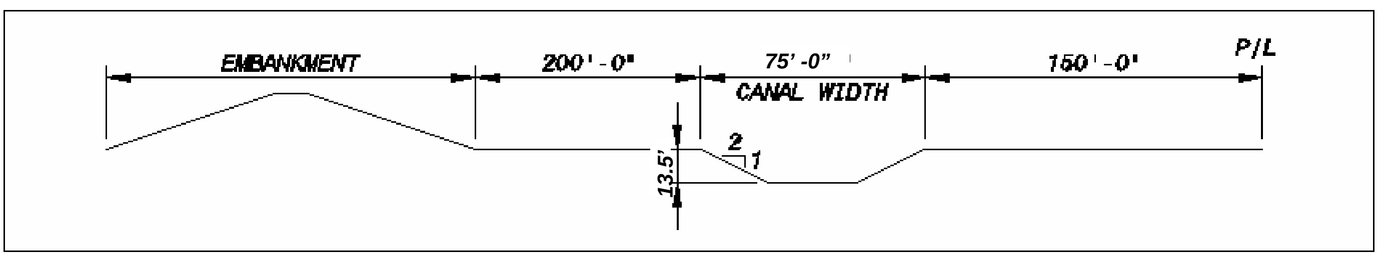
The configuration of the EAA Reservoir A-1 embankment and seepage canals directly affects the total amount of storage for EAA Reservoir A-1. In order to achieve storage of 190,000 acre-feet, setback requirements were balanced with the total area available to meet this requirement. The evaluation of various setbacks are presented in greater detail with respect to SFWMD, USACE, and USFWS requirements, construction considerations, cost, and other factors in Appendix 8-11 and Appendix 8-12. Setbacks for each portion of the EAA Reservoir A-1 are summarized in the following Sections, and result in a configuration that provides storage in EAA Reservoir A-1 slightly more than the 190,000 acre-feet required at a depth of 12 feet and a reservoir footprint area of approximately 16,000 acres. Except as noted, setbacks identified are applicable for either an earthen embankment or an RCC dam.

12.1.1 North Boundary and North Portion of West Boundary (Portion North of Future EAA Reservoir A-2)

See Figure 12.1-1

- 150-foot setback from EAA Reservoir A-1 boundary to the seepage canal, in part to provide for future construction access to the EAA Reservoir A-2 site
- 75-foot wide seepage canal
- 200-foot setback from seepage canal to the outside toe of the embankment for construction stockpiling and future wetland areas
- 300-foot setback from the inside toe of the embankment to the internal borrow excavation

Figure 12.1-1 North Boundary Setbacks



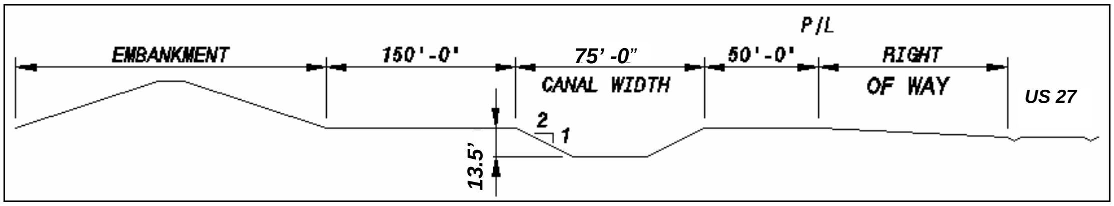
12.1.2 East Boundary (Portion Adjacent to US 27)

See Figure 12.1-2

- 50-foot setback from US 27 right-of-way to the seepage canal
- 75-foot wide seepage canal

- 150-foot setback from seepage canal to the outside toe of the embankment for construction stockpiling and future wetland areas
- 300-foot setback from the inside toe of the embankment to the internal borrow excavation

Figure 12.1-2 East Boundary Setbacks



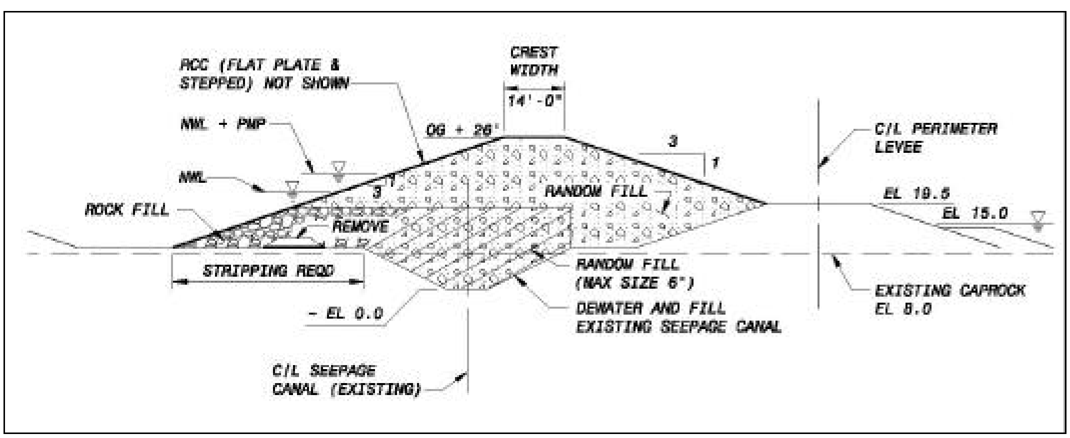
12.1.3 South Boundary and South Portion of West Boundary (Portion Adjacent to the STA-3/4 Supply Canal and Along Future EAA Reservoir A-2)

For an earthen embankment, a configuration along the STA-3/4 Supply Canal would be utilized that provides cost-savings and additional storage. Rather than setting back the embankment from the EAA Reservoir A-1 boundary, the embankment would tie in with the northern levee of the seepage canal. This embankment, as shown in Figure 12.1-3, is discussed in further detail in Section 8 of this BODR. The following setback would still apply to this embankment.

- 300-foot setback from the inside toe of the embankment to the internal borrow excavation

For an RCC dam along the STA-3/4 Supply Canal, setbacks as outlined for the east boundary would apply, and any cost-savings or additional storage would be negated.

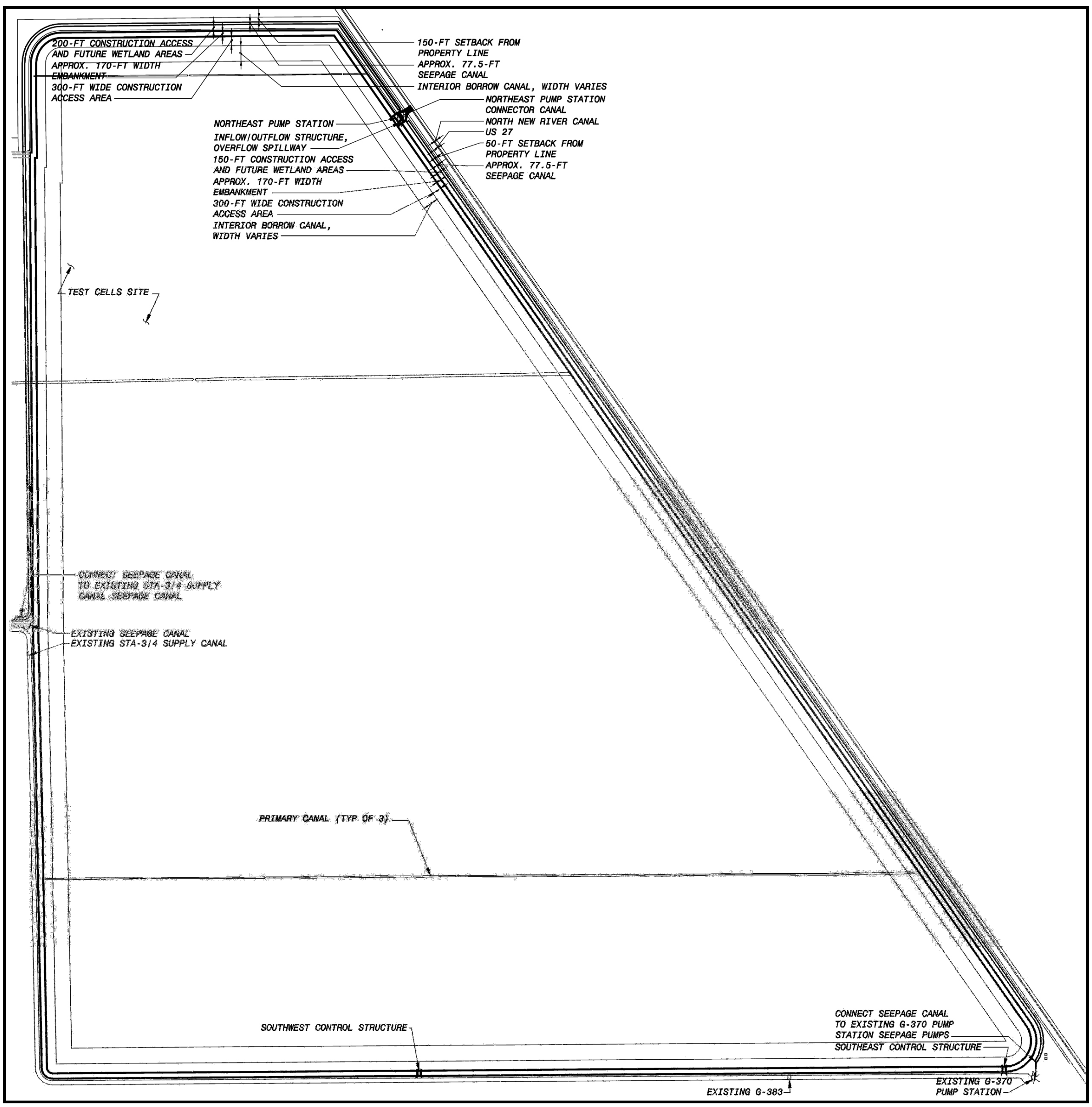
Figure 12.1-3 Earthen Embankment along STA-3/4 Supply Canal



Curved corners provide an additional benefit as discussed in Appendix 8-11 and 8-12 of this BODR. Curved corners will be utilized in the northwest and southeast corners of EAA Reservoir A-1. Both the northwest and southeast corners will be curved at a radius that aids construction of the embankment. Due to the acute angle in the southeast property corner, an embankment configuration that parallels the property line adds little additional storage. Therefore, attention will be given to cost when selecting the embankment radius in the southeast corner. Additionally, the configuration of the southeast corner will be sufficient to prevent relocation of existing facilities, including existing helipads.

For an earthen embankment that incorporates the STA-3/4 Supply Canal, curved corners are not feasible, and therefore will not be utilized in the southwest corner of EAA Reservoir A-1. However, for an RCC dam, the embankment will be setback from the STA-3/4 Supply Canal, and curved corners will be utilized at a radius that aids construction of the embankment. In the northeast corner, curved corners are not feasible due to the location of the new northeast pump station, and therefore, will not be utilized. An overall site plan is provided as Figure 12.1-4.

Figure 12.1-4 Overall Site Layout



12.2 SITE ACCESS AND ROADWAYS

General access to the EAA Reservoir A-1 and associated structures will be limited to SFWMD staff and guests. Public access to the EAA Reservoir A-1 will only be allowed through designated public access points, primarily along US 27. Public access locations will be designed to support nature based recreation in accordance with SFWMD standards.

The Project is located in an agricultural area and access to the Project site is limited to one improved state highway, US 27, and existing gravel and dirt service roads created either by the agricultural interests or by the SFWMD operations and maintenance staff. US 27 is a north-south trending four-lane divided highway that borders the entire east side of the EAA Reservoir A-1 and the site for the new northeast pump station. This is a major traffic route for transportation from the Fort Lauderdale area to the central Florida area, and is also a hurricane evacuation route. It is anticipated that this will be the primary access road to be used by the contractor during construction. After the project is complete, US 27 will provide the main access to the EAA Reservoir A-1 and the northeast pump station.

The EAA Reservoir A-1 Project is bordered on the north side by an existing gravel service road used by the agricultural interests. This service road has access to US 27 by way of an existing intersection. The EAA Reservoir A-1 is bordered on the west side by an agricultural area and by the Holey Land Wildlife Management Area. A SFWMD service road exists along the southern half of the EAA Reservoir A-1 Project area’s west side, and continues east along the southern boundary. This service road provides SFWMD vehicle access from the G-372 pump station to the G-370 pump station. Currently there is a canal between the SWFMD service road and the EAA Reservoir A-1 site.

Permanent access to the EAA Reservoir A-1 will include perimeter roads constructed completely around the top of the EAA Reservoir A-1 embankment and completely around the toe of the embankment between the embankment and the seepage canal. The road along the top of the embankment will be used for access to control structures and for inspection of the inside of the EAA Reservoir A-1. The road around the embankment between the embankment and the seepage canal will be used for inspection of the exterior embankment slope. Access to these perimeter roads will be provided in four locations:

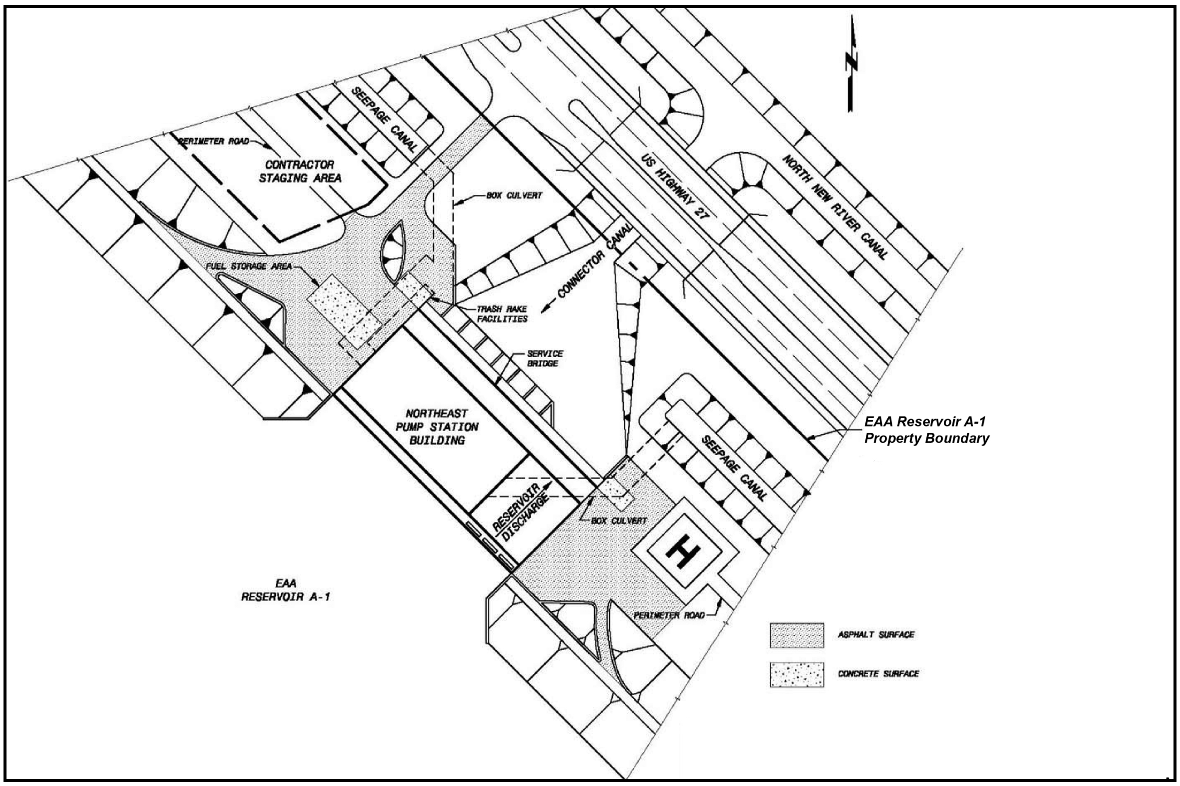
- At the northeast corner of the EAA Reservoir A-1 near the intersection of the existing service road and US 27
- At the new northeast pump station
- On the west side of the EAA Reservoir A-1 near the northeast corner of the Holey Land Area
- At the existing G-370 pump station

Access to cross the seepage canals and ramps to the top of the embankment will be provided at these locations in accordance with Section 12.3 hereafter. All vehicular roads will be designed, as a minimum, to meet SFWMD standards, as set forth in the DCMs.

Access ramps to the inside of the EAA Reservoir A-1 will be provided near each of the four corners of the EAA Reservoir A-1 and at the new northeast pump station. The access ramps will include boat ramps and will allow water quality sampling along with inspection and maintenance of the inside of the EAA Reservoir A-1.

A new intersection on US 27 will provide access to the new northeast pump station. This new intersection will be located at least 1,300 feet from the closest existing intersection, will include acceleration and deceleration lanes, and a median crossing. An entrance road to the northeast pump station will connect US 27 to the northeast pump station structure, fuel storage area, and parking areas. See Figure 12.2-1 below for an illustration of the planned entrance road and associated parking areas. Security issues for the northeast pump station will be considered during detailed design.

Figure 12.2-1 Northeast Pump Station Site Plan



Access to the new outlet gate structures in the south embankment along the Supply Canal will be provided by the perimeter road located along the top of the embankment and by the existing perimeter levee road located along the toe of the embankment’s exterior slope as shown in Figure 12.1-3.

There are other east-west service roads within the boundary of the EAA Reservoir A-1 that have intersections with US 27. These service roads will be removed during the course of the EAA Reservoir A-1 construction, but they will most likely provide construction contractors access to the EAA Reservoir A-1 site. Staging areas for the construction of the EAA Reservoir A-1 will be determined by the construction contractors, and may move as construction work progresses. The construction contractor will be limited to entering the EAA Reservoir A-1 Project site from US 27.

It will be the responsibility of the construction contractors to coordinate with FDOT regarding the maintenance of traffic during construction.

Provisions for maintaining drainage and irrigation for agricultural properties are addressed in the construction coordination section.

12.3 BRIDGES

The new northeast pump station moves water from the NNRC into the EAA Reservoir A-1 and visa versa. The NNRC runs parallel to and on the east side of US 27. The EAA Reservoir A-1 and northeast pump station are located on the west side of US 27. Therefore, a new connector canal will be required to connect the NNRC to the northeast pump station. A new highway bridge will be required to carry US 27 traffic across the connector canal. The new bridge will be designed to FDOT standards. A multi-span concrete slab bridge is anticipated. A bridge development report will be prepared as part of the 30 percent design.

Bridges will also be required across the seepage canals at vehicular access points to the EAA Reservoir A-1. These bridges may be of the culvert type.

12.4 STORMWATER CONTROL/SITE DRAINAGE

12.4.1 During Construction

The size and nature of this Project causes concern for managing the stormwater runoff during construction. A conceptual Stormwater Pollution Prevention Plan (SWPPP) will be required as a part of the contract documents. The objective of the SWPPP will be to prevent erosion where construction activities are occurring, prevent pollutants from mixing with stormwater, and prevent pollutants from being discharged by containing them on-site, before they can affect the receiving waters. The contractors will be required to prepare and submit a comprehensive SWPPP that will be tailored to their sequence of construction. The contractor will be provided conceptual plans, guidelines, and criteria so that detailed drainage plans for all phases and sequences of construction can be prepared.

12.4.2 Permanent Construction

The site grading around the new northeast pump station will include provisions for capturing and treating, where necessary, the stormwater runoff. Long term site drainage along the north and east borders will be directed to the EAA Reservoir A-1 seepage canal. Long term site drainage along the STA-3/4 Supply Canal will be directed to the Supply Canal. Long term site drainage along the west border will either be directed to the EAA Reservoir A-1 seepage canal or to the Miami Canal via existing canals. Stormwater calculations and facilities will be designed to comply with local and State guidelines and regulations.

Maintenance of irrigation during and after construction to the agricultural areas west of the EAA Reservoir A-1 site is discussed in Section 22 – Construction Coordination of this BODR.

12.4.3 Utilities

12.4.3.1 Electric Power

12.4.3.1.1 Florida Power & Light Overhead Transmission Lines

Electric transmissions lines are located along the US 27 highway easement. Electric power for the northeast pump station can be obtained from those lines. There are no overhead transmissions lines that will need to be removed or relocated for the EAA Reservoir A-1 project.

12.4.3.1.2 FPL Overhead Primary Lines

There is an existing Florida Power & Light (FPL) medium voltage overhead primary line within the EAA Reservoir A-1 Project footprint. This line provides electric power to an agricultural pump station located on the west side of the EAA Reservoir A-1. The line will need to be removed and, if a decision is made such that this pump station must stay in operation during and after construction is completed, electric power will need to be restored to the agricultural pump station. Black & Veatch will coordinate with the utility owners and obtain input from them regarding utility relocation for design purposes. Demolition of the existing utilities within the EAA Reservoir A-1 footprint is understood to be the responsibility of the utility owner and will be coordinated with the construction contractor.

The new northeast pump station will require a primary power line connected to the transmission line in the US 27 easement.

The new outlet gate structures located along the south embankment of the EAA Reservoir A-1 will require a primary power line. Sources for this power line to be investigated include connection to:

- The transmission line in the US 27 easement
- The existing primary line supplying power to the STA-3/4 inlet gates
- The existing primary line supplying power to the G-370 pump station

SECTION 13

MECHANICAL DESIGN

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13. MECHANICAL DESIGN

13.1 *INTRODUCTION*

This Section describes mechanical equipment requirements for the EAA Reservoir A-1. Primary mechanical needs include equipment that will be installed in the new northeast pump station and inlet and outlet structures between the EAA Reservoir A-1 and both the NNRC and the STA-3/4 Supply Canal. In addition, although there are no modifications proposed to the existing G-370 and G-372 pump stations as a result of the EAA Reservoir A-1, a discussion is included regarding modifications proposed in conjunction with the construction of the future EAA Reservoir A-2 for cost comparison purposes.

13.2 *EXISTING G-370 AND G-372 PUMP STATIONS*

Several alternatives for modifying the existing G-370 and G-372 pump stations were developed and are summarized in Appendix 6.11, Pumping and Discharge Facilities Technical Memorandum. Included were modifications to allow pumping into the EAA Reservoir A-1 to water surface elevations varying from the operating level of the STA-3/4 Supply Canal to a full EAA Reservoir A-1 depth at elevation 20.6 NAVD88.

13.2.1 Existing Station Capacity

The G-370 pump station contains three vertical pumps and the G-372 pump station contains 4 vertical axial flow pumps, all rated at 925 cfs at a design discharge elevation of 13.6 NAVD88 for the G-370 pump station and of 15.6 NAVD88 for the G-372 pump station. All pumps are driven by internal combustion engines. Each engine is connected to its associated pump through a right-angle gear reducer.

Support systems provided for the engines driving the main pumps include:

- Fuel oil supply system consisting of a fuel oil receiving pump, storage tanks, and transfer pumps for transferring fuel oil to day tanks at the engines
- Lube oil supply system including a storage tank and transfer pumps provided to automatically make up oil lost from the engine crankcase
- Waste lube oil collection system provided to collect oil from the engines and engine-generators
- Cooling water supply system for the engines and the gear reducers
- Compressed air system provided for starting the engines

A fresh water supply system provides water for water lubricated pump bearings and for a potable water supply system.

The existing pumps can operate to slightly higher discharge elevations, but at a reduced capacity. Each pump in the G-370 pump station can pump to discharge elevation 16.6 NAVD88 at a rated capacity of about 780 cfs for a pump station capacity of 2,340 cfs. Pumps in the G-372 pump station can pump to discharge elevation 17.6 NAVD88, also at a pump capacity of 780 cfs or a total pump station capacity of 3,120 cfs. Because of engine output capacity limitations, the pumps cannot pump to higher discharge elevations without modification.

No modifications to the existing pump stations are proposed for this phase of the EAA Reservoir A-1 Project. The stations will operate in several different modes:

- EAA Reservoir A-1 water level less than 16.6 feet NAVD88, EAA Reservoir A-1 filling: The Supply Canal will operate at the same elevation as the EAA Reservoir A-1. Both pump stations will pump to the Supply Canal and flow can be directed to either the EAA Reservoir A-1 or STA-3/4.
- EAA Reservoir A-1 water level greater than 16.6 feet NAVD88, EAA Reservoir A-1 filling: Gates to the EAA Reservoir A-1 will be closed and the Supply Canal will operate at elevation 13.6 NAVD88 or less. Discharges to the Supply Canal would be at rated capacity and would be directed directly to STA-3/4.

- EAA Reservoir A-1 water level greater than 13.6 feet NAVD88, EAA Reservoir A-1 discharging: The Supply Canal would operate at elevation 13.6 or less. EAA Reservoir A-1 discharges would be to the Supply Canal by gravity. Both pump stations would be available to pump to the Supply Canal at rated capacity, and flows would be directed to STA-3/4.
- EAA Reservoir A-1 water levels less than 13.6 feet NAVD88, EAA Reservoir A-1 discharging: Gates between the Supply Canal and EAA Reservoir A-1 would be closed. EAA Reservoir – A-1 discharges would be routed to the G-370 pump station through the NNRC. Both pump stations would be available to pump to the Supply Canal at rated capacity, and flows would be directed to STA-3/4.

13.2.2 Future Modifications

The proposed Project includes modifications to the existing pump stations to provide capacity to pump to full EAA Reservoir A-1 depth when the EAA Reservoir A-2 is brought on line.

The existing engines at both pump stations operate at 720 rpm and are naturally aspirated. The engines can be modified to provide additional power by increasing the engine speed and with the addition of a turbocharger on the engine. These two modifications together can potentially increase the engine output capacity by approximately 65 percent at the G-370 pump station and by approximately 100 percent at the G-372 pump station. This additional power can be used to increase the pumping head available, permitting the pumping units to pump to an EAA Reservoir A-1 elevation of 20.6 NAVD88.

However, in order to take advantage of the available additional horsepower, a number of modifications to the engine auxiliary systems and the pump station must be made. These include just about every mechanical system within the pump station related to the operation of the engine. Many of these systems may have to be upgraded or replaced. Equipment and systems potentially affected by increasing the engine horsepower rating and pump head include the following:

- Engine Exhaust System – Increased output capacity of the engine will increase the volume of exhaust gas. This will require replacing the existing exhaust system with larger exhaust silencers and larger diameter exhaust pipe. Turbocharging the engines requires significant changes to the support of the exhaust system.

- Fuel System – Increasing the output capacity of the engines will increase the fuel consumption, however it is anticipated that the existing capacity of the main fuel storage tanks are still suitable to provide the desired five days of fuel storage.

- Engine Jacket Water, Engine Oil, and Gear Reducer Cooling System – Cooling is critical for proper operation of the engine. Increasing the output capacity will result in increased cooling requirements.

- Building Ventilation and Filtering – Additional combustion air supply and additional ventilation for cooling the spaces in the building is required.

- Drive Shaft – Increasing the engine rating and increasing the engine drive speed will result in increased torque and stress on the drive shaft from the engine to the gear reducer and it will need to be replaced with a larger shaft.

- Gear Reducer – The existing right angle gear reducer horsepower rating is inadequate for any increase in engine rating and would need to be replaced.
- Pump Shaft and Couplings – All options which include increasing engine rating will require replacement of the pump shaft.
- Engine-Generators – The required increase in motor sizes for auxiliary system loads will require replacement of the existing engine-generators with larger units.
- Exhaust Emission Permits – A new Title 5 emission permit will be required.

Modification Options A through F were evaluated and are presented below. The options are also summarized and the capacities for the G-370 pump station are presented on Figure 1 in Appendix 6-11. Options B through F provide modifications so that the existing pump stations can pump to full EAA Reservoir A-1 depth. For all options listed, the G-370 pump station would retain 2,775 cfs capacity when pumping directly to the STA-3/4. Although the same array of modifications could be applied to the G-372 pump station, it is assumed that only options E or F (see below) would be used to modify that pump station because they result in capacities of 3,700 cfs. Any of the other options would result in a reduction in pump station capacity and would diminish the flood protection capability of the pump station.

- **Option A – No Change** - The characteristics of the pump performance curves for the type of propellers used in the G-370 pump station and the G-372 pump station is that the horsepower requirement increases with pumping head and decreasing flow rate. For this reason there is a limit to how high the existing pumps can pump before exceeding the capacity of the engines. For this option there are no changes to the pump stations considered, and water is diverted from the STA-3/4 Supply Canal into the EAA Reservoir A-1 up to the design water surface of the STA-3/4 Supply Canal. Above this level, the engines will be at or above the nameplate ratings of engines and can not be operated. Therefore, up to the design water surface of the STA-3/4 Supply Canal, water can be delivered to the STA-3/4 and to the EAA Reservoir A-1. Above this elevation, water can only be delivered to STA-3/4.
- **Option B - New Propeller** - For this option, the existing propellers are replaced with new propellers that pump to higher heads at lower flows. Pump capacity at rated head is 340 cfs; the G-370 pump station capacity is 1,020 cfs.
- **Option C - Increase Engine and Pump Speed** - The engines provided for the pump station are rated at 720 revolutions per minute (rpm). However, according to the manufacturer they can be operated at 900 rpm. By increasing the engine speed, the output power of the engines can be increased permitting the pump propeller to operate at higher speed and therefore capable of pumping to higher head levels. Pump capacity at rated head is about 625 cfs; the G-370 pump station capacity is 1,875 cfs.
- **Option D - Increase Engine and Pump Speed and Replace Propeller** - To further increase the capacity of the G-370 pump station, a new propeller design was investigated that permits the pump station to pump more water at the design EAA Reservoir A-1 elevation for the same horsepower required in Option C. Pump capacity at rated head is about 740 cfs; the G-370 pump station capacity is 2,220 cfs.

- **Option E - Increase Engine and Pump Speed and Turbo-Charge Engine –** Turbocharging the engine can significantly increase horsepower to pump at or near the current design flow capacity of the station to the design water level of the EAA Reservoir A-1. Pump capacity at rated head is 925 cfs; the G-370 pump station capacity is 2,775 cfs and the G-372 pump station capacity is 3,700 cfs.
- **Option F - Increase Engine and Pump Speed, Turbo-Charge Engine, and Replace Propeller** - A different propeller design was investigated that would increase the pump efficiency at the higher EAA Reservoir A-1 levels resulting in higher pumping rates. Pump capacity at the rated head is 925 cfs; the G-370 pump station capacity is 2,775 and the G-372 pump station capacity is 3,700 cfs.

13.2.3 Pump Station Discharge Modifications

For future alternatives where the pump stations pump into a full EAA Reservoir A-1, the water surface will be above the pump discharge sill elevation resulting in backflow of water through the pumps when they are stopped. The improvements to the G-370 pump station shown in Figure 13.2-1 below includes flap gates on each pump discharge to prevent backflow of water through a pump when it is not operating. Also included is a gate structure divided into three sections, one for each pump, to allow accessing the flap gates for each pump individually for maintenance. Other improvements required include raising the level of the embankments on the discharge side of the pump station and gate structures at the inlet to the EAA Reservoir A-1 and at the Supply Canal.

Two new gate structures will control the flow of water to the EAA Reservoir A-1 and to the STA-3/4 Supply Canal. The EAA Reservoir A-1 gate structure would act as the EAA Reservoir A-1 wall. The STA-3/4 Supply Canal gate structure would allow control of flow to the STA-3/4 Supply Canal. When the gates are closed, pump station discharge flow would be directed through the gate structure to the EAA Reservoir A-1 and when opened, the flow directed to the STA-3/4 Supply Canal. These gates would also enable water from the EAA Reservoir A-1 to flow in the reverse direction through the gate structure and be directed to the STA-3/4 Supply Canal.

13.2.4 Pump Station Modifications Conceptual Opinion of Probable Cost

Table 13.2-1 presents the conceptual opinion of probable cost for the mechanical modifications and the discharge modifications to the G-370 and G-372 pump stations. Gate structures vary depending on the pumping and discharge facilities alternative considered, and are therefore, not included. The breakdown for the probable costs presented in Table 13.2-1 is presented in Appendix 6-11.

Figure 13.2-1 G-370 Pump Station Discharge Modifications

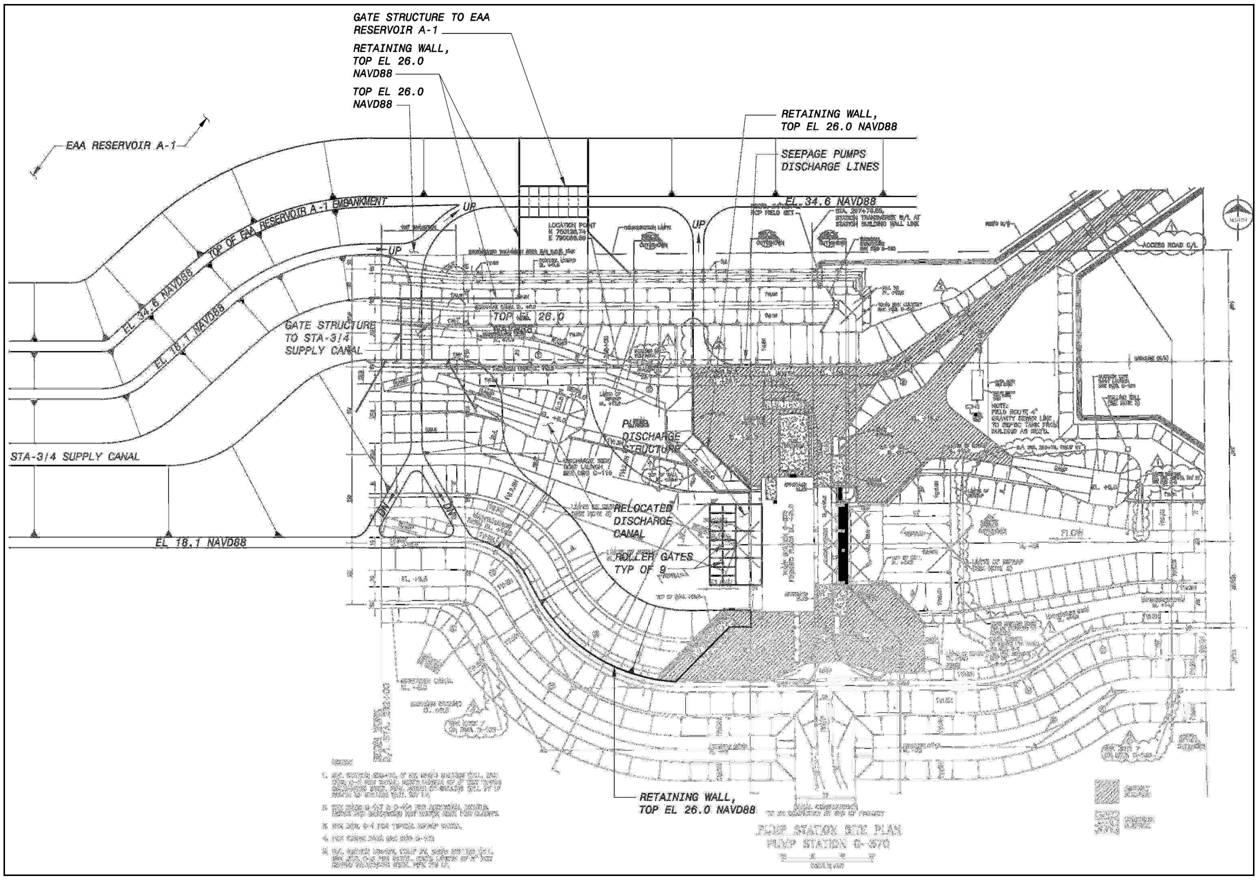


Table 13.2-1 Pump Station Discharge Modifications Conceptual Opinion of Probable Cost

	Cost (Million $)	**Indirect Costs** (Million $)	**Total** (Million $)
Mechanical Modifications – G-370 Pump Station			
Option A	0	0	0
Option B	1.4	1.3	2.7
Option C	5.2	4.5	9.7
Option D	6.0	5.3	11.3
Option E	7.4	6.6	14.0
Option F	8.3	7.4	15.7
Mechanical Modifications – G-372 Pump Station			
Option A	0	0	0
Option B*	1.8	1.6	3.4
Option C*	6.3	5.6	11.9
Option D*	7.5	6.5	14.0
Option E	10.0	8.9	18.9
Option F	11.2	9.9	21.1
Discharge Modifications			
G-370 Pump Station	6.4	5.7	12.1
G-372 Pump Station	8.5	7.6	16.1
Seepage Pump Modifications			
G-370 Pump Station	0.5	0.4	0.9
G-372 Pump Station	0.6	0.6	1.2
*Note: Not recommended as a viable option			

13.3 *NORTHEAST PUMP STATION*

13.3.1 Design Criteria

The northeast pump station will be located at the northeast corner of the EAA Reservoir A-1 and shall connect to the NNRC via a connector canal that will include a highway bridge for U.S 27 which parallels the NNRC to its west. The station's function is to pump basin runoff and Lake Okeechobee releases from the NNRC to the EAA Reservoir A-1. The northeast pump station's pumps shall have the capability of lifting the water from the connector canal's design low water stage to the proposed EAA Reservoir A-1's maximum stage of 20.6 NAVD88. During detailed design, configuration options will be considered that allow water to be pumped out of the EAA Reservoir A-1 to the NNRC when stage levels drop below 10.5 NAVD88 as discussed in Section 6.6. See Figures 13.3-1 and 13.3-2 for preliminary illustrations of the northeast pump station.

Figure 13.3-1 Northeast Pump Station Building Section

Not to scale

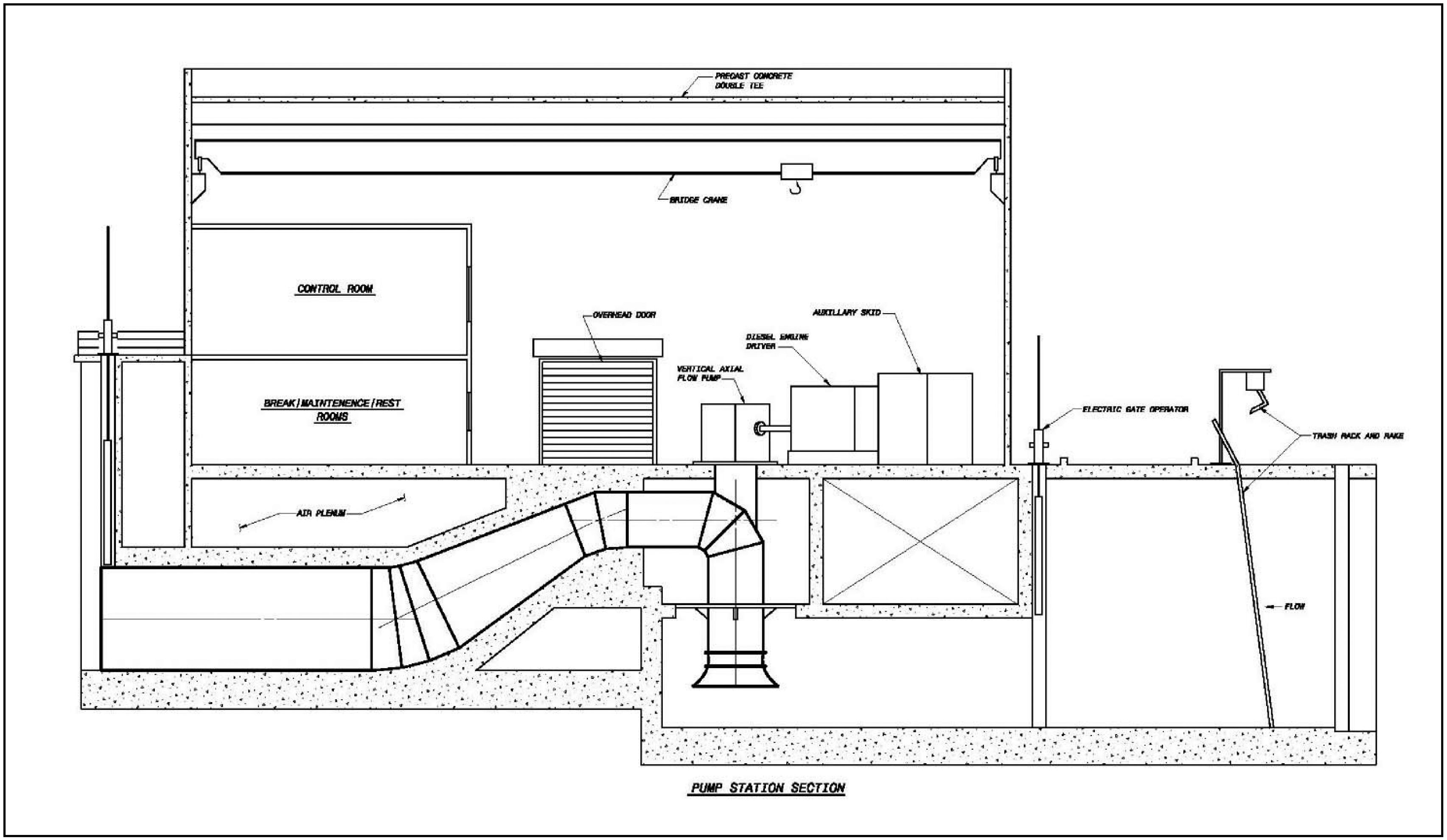
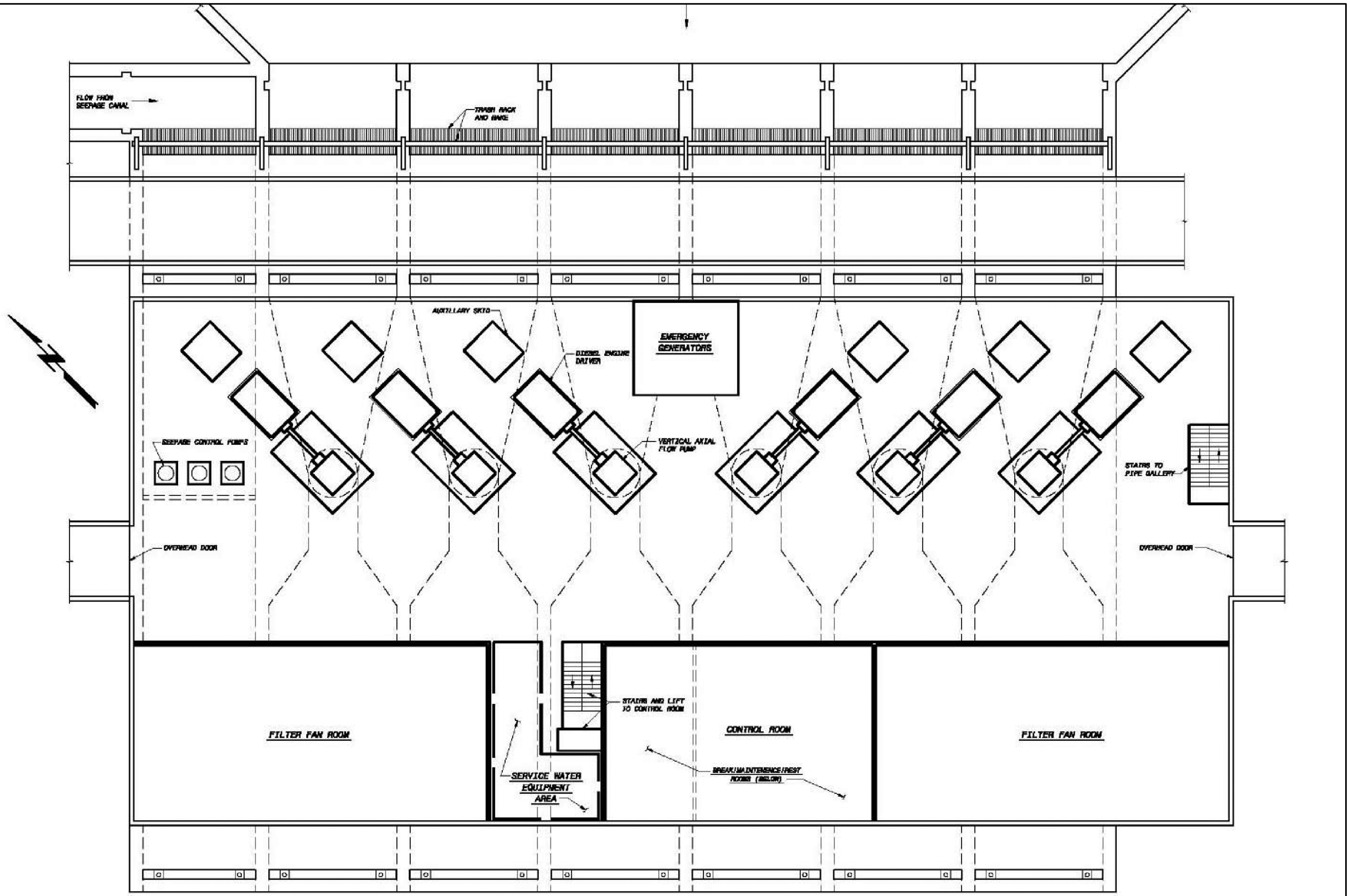


Figure 13.3-2 Northeast Pump Station Floor Plan
(not to scale)



PUMP STATION FLOOR PLAN

Note: Configuration modifications necessary for pumping out of the EAA Reservoir A-1 when stage levels drop below 10.5 will be considered during detail design.

13.3.2 Equipment: Northeast Pump Station

- Capacity: 3,600 cfs
- Number of pumps: total pump bays – 6
- Pump capacity: 6 – 600 cfs pumps
- Design static head range, min/max: 1.75/15.0
- Pump arrangement: vertical, axial flow
- Pump intake: suction bell
- Pump driver: engine-driven with right-angle gear drive
- Engine fuel supply: diesel
- Discharge arrangement: Through the embankment with dual roller gates
- Trash racks: bar racks with picker style traveling rake

13.3.3 Protection Elevation

The operating floor elevation at the northeast pump station should limit the possibility of damage, caused by flooding, to the pump equipment. There are two fundamental methods for a discharge configuration into the EAA Reservoir A-1: 1) Over-the-Embankment and 2) Through-the-Embankment Discharge. Those methods are discussed in greater detail in a subsequent section of this document. For the over-the-embankment discharge configuration, the operating floor elevation is dependent on the maximum water elevation that can be expected in the EAA Reservoir A-1. Due to wind set-up and wave run-up, the top of the proposed EAA Reservoir A-1 embankment is set at 26 feet above existing grade (approximately elevation 34.6 NAVD88), or approximately 14.0 feet above the normal maximum pool stage of the EAA Reservoir A-1 of 20.6 NAVD88. Therefore, for the over-the-embankment discharge configuration, the operating floor elevation shall be 34.6 NAVD88. For the through-the-embankment discharge configuration, the operating floor plan will be set below the maximum water elevation in the EAA Reservoir A-1. Therefore, a flood wall shall be provided to protect the station. However, the clearances necessary for below the base plate pump discharge and its coupling may be the critical dimension and must be coordinated with the pump manufacturer.

13.3.4 Connector Canal Considerations

The connector canal to the northeast pump station shall intersect the NNRC at 90 degrees and proceed a short distance west under a bridge crossing for U.S. 27 and to the northeast corner of the EAA Reservoir A-1. The flow approaching the pump intake should ideally be steady and uniformly distributed both laterally and vertically. The approach flow asymmetry and unsteady flow conditions are caused by the geometric layout of the connector canal and the intake. In practice it is not possible to completely eliminate non-uniform or unsteady flow conditions. The ideal hydraulic condition is for the connector canal to be in line with the intake centerline. The northeast pump station wingwalls should be at an angle of no more than 10 degrees from the centerline. In addition, the northeast pump station connector canal will require a decrease in bottom elevation to the intake. This transition slope should not exceed 10 degrees. It should also be noted, a surface drop can occur across a partially blocked trash rack, or whenever the pumps have lowered the water level in the sump to the point at which all pumps are about to be switched off. Therefore, the path between the sump entrance and the pump inlets must be sufficiently long for the air bubbles to rise to the surface and escape before reaching the pumps. The energy of the falling water should be dissipated sufficiently so that excessively high and irregular velocities do not occur within the sump. This can be accomplished with properly designed and placed baffle walls.

13.3.5 General Mechanical Arrangement Alternatives

13.3.5.1 Station Design Considerations and Criteria

One objective of the pump selection and the northeast pump station design is to achieve the lowest total head to provide the smallest driver, and therefore, the lowest energy cost. However, flood control pump stations are operated relatively infrequently, maybe less than 1,500 hours per year. Therefore, energy savings is not as critical a design parameter as it would be for water supply or other applications where the pumps run continuously. However, the EAA Reservoir A-1 pump station shall have the reliability considerations afforded a flood control station but shall also be assumed to see continuous duty as a water supply station. Efficiency of its mechanical equipment as well as the facility design will be an important factor in its overall design.

13.3.5.2 Design Life

There are a number of USACE references in regard to pump station service life. According EM-1110-2-3104, EM-1110-2-3105, and the SFWMD’s Major Pump Station Engineering Guidelines, the design life of a pump station is 50 years. It is anticipated the mechanical equipment will required rehabilitation or replacement at least once during the 50-year service life. The engines and pumps will operate intermittently and with proper preventative maintenance the engine should have a 25-year service life. Pump equipment shall be designed and manufactured to ensure a long service life.

The engineering regulation ER 1110-2-8159, “Engineering and Design - Life Cycle Design and Performance” defines the engineering policies for selection of all systems, components, and materials for civil works projects on the basis of their long term performance. This regulation requires the design engineer for civil works projects to use life cycle design as the basis for selection of all project elements such as materials, structural systems, mechanical equipment, and site appurtenances. Life cycle cost analysis is an important and increasingly standard method to compare alternatives of major hydraulic facilities during the conceptual development of the project. Therefore, for the BODR, a life cycle cost analysis shall be provided of the proposed alternatives for evaluation and recommendation of the selected alternative.

13.3.5.3 Reliability and Cost Considerations

Flood protection pump stations should be considered emergency facilities. Equipment and power supply are specified and selected primarily on the basis of reliability under emergency conditions. Pump stations are one of the more vulnerable features of a flood protection project. Dependability must be a primary consideration during the design and selection process.

The designer needs to be aware of cost, but, because of the typically infrequent operation of flood control pump stations, efficiency can sometimes be sacrificed to a degree in favor of equipment with a lower initial cost. However, for large pump stations with frequent usage, higher pump efficiencies can lower the installed horsepower requirements and reduce capital and operating costs significantly. Therefore, the engineer should consider higher efficiency equipment on a life cycle basis. Auxiliary systems should be minimized, the more things that can go wrong will. Refinements that make no realistic contribution to the usability or dependability should be avoided. The equipment selected must be rugged, reliable, and well suited for the service. The station should be of a robust design that is sized to house and support the equipment. Since the station can potentially be in service during extreme weather conditions when commercial power is not available, the pumps are engine driven and the station auxiliaries have backup power by engine driven generators.

Water supply, seepage return, or other pumping systems not associated with flood control or drainage do not have the same demand for high reliability. Their operation is not critical to the health and well being of the public, down time can be tolerated with repairs addressed in non-emergency mode. The drivers can be electric motors where power is available, and power outages have less impact.

13.3.5.4 Number of Pumps

An initial step in the development of the station design is the determination of the suitable number of pumps to meet the flow demand. The operational model typically indicates a logical number of pumps of various rated capacities that when combined in parallel operation, satisfy the operational flow rate requirements. Traditionally, a minimum of two pumps should be provided with three pumps typically specified unless the foundation conditions warrant a larger number of units. In flood control stations, there should be a one pump redundancy in case of pump failure. Generally, the number of pumps should be kept to the minimum to reduce equipment capital cost, maintenance cost, and minimize the footprint of the intake to reduce construction costs. However, it may be more cost effective to have a greater number of smaller pumps to reduce the depth of the intake as well as reduce foundation loads. Smaller pumps also provide for more operational flexibility to address less than maximum flow operations. Considerations for selection of the number of pumps include:

- Station cost
- Station reliability/availability
- Maintenance cost
- Energy cost
- Operational demands and flexibility
- Foundation loads
- Driver horsepower
- Availability of pump models
- Intake depth

The historic hydrologic data summary presented earlier indicated the need for small incremental (500 cfs) capacity capability for the station. However, this historic data most probably will not represent the future conditions with the enlargement of the NNRC and potential inter-basin transfer of runoff discharges. There are no operational criteria available to determine a needed incremental flow rate to the EAA Reservoir A-1 developed to date. The EAA Reservoir A-1 has significant storage capacity to attenuate large inflows. Therefore, because of this storage volume, there is no need for small capacity pumps to maintain a target operational stage within the EAA Reservoir A-1. Therefore, a basic assumption that needs to be made at this point is there is no apparent reason for the station to have pumps of different capacities. This is a significant positive aspect in regard to construction and installation costs requiring identical intakes, operating floor equipment layout, and equipment and auxiliary installation. Having identical drivers, reducers, and pumps reduce the spare part inventory, operator training, and the necessary operator experience, making the pump station significantly simpler to operate.

Given this simplification of equal pump capacities, it now becomes a decision of the number of the pumps. There are two cost items of the pump station that have a major influence on the pump station's total cost and represent a good indicator of the least cost alternative, the pump intake, and the pump equipment and its auxiliaries. The incremental cost of equipment does not change in a linear manner with decreasing capacity. In fact an engine model may be applicable to two alternatives since manufacturers produce engine models to address a range of horsepower requirements. This is also true of the pump and reduction gears. Therefore, going from four pumps to five may not change the model of the equipment, only the design of some of its components, i.e. shaft size and propeller geometry. This is true with the auxiliary systems as well where there may be even less selection in service ratings for this support equipment, i.e. service water cooling and lubrication systems. The small incremental reduction in equipment cost is more than off-set by the increase in construction and labor costs. The more systems to install, the labor cost increases almost linearly. The time to install a 150 gallon day tank vs. a 125 gallon day tank, or a 1-inch diameter fuel line vs. a 1.5-inch line is more or less the same. Therefore, the more pump systems and their auxiliaries to be supplied and installed the greater the total cost of this item of work. The axiom in regard to equipment, the fewer the number of pumps the lower the cost.

For this BODR phase of the Project, one objective is to determine the optimum size of the northeast pump station's pump capacities based on a life cycle cost analysis. Since the EAA Reservoir A-1's future operational scenarios have yet to be determined, it was considered prudent to include and array of pump numbers from the traditional optimum number of three to six. Therefore, alternative pump station designs considered included multiple pump combinations. The analysis shall, therefore, provide useful information for the future design given the need for a smaller incremental inflow rate to the EAA Reservoir A-1.

13.3.5.5 Pump Alternatives

With the determination of the number of pumps and their respective rated capacities, the designer can select a pump type that satisfy the basic flow and head requirements. The head conditions for the proposed northeast pump station alternatives are in the low head range. Axial flow pumps deliver large capacity flows at low heads and have specific speed in the range of 10,000 to 15,000+.

$$Ns = Nt\,(Q^{0.5})/H^{0.75}$$

Ns = pump specific speed

Nt = pump rotative speed, (rpm)

Q = flow at the Best Efficiency Point (BEP) in gallons per minute (gpm)

H = head at the BEP, (feet)

This pump type also maintains good efficiency at a constant speed over a considerable greater range of heads compared to the lower specific speed pumps and is particularly applicable to variable head duties. The suction lift of this type pump is negligible. These pumps are typically used for flood control, water supply, or drainage applications where low head conditions exist and high capacity is required. They are capable of low heads from six to 20 feet with capacities up to 500,000 gpm, (1,110 cfs).

The advantages of axial flow pumps include:

- Ability to address large flow requirements
- Relatively inexpensive
- The pump can be easily changed by changing an impeller, to satisfy a new duty point
- Pull out designs allow the rotating element be quickly and easily removed for inspection
- For long pumps the pull out design reduces crane capacity requirements

The disadvantages:

- Not capable of heads over 20 feet
- No suction lift
- Slow rotative speed of the pump requires a gear transmission to reduce the shaft speed of the driver or use of a slow speed pump
- Water lubricated bearings result in higher maintenance requirements

The low head conditions of the system designs of all alternatives of the northeast pump station will require axial flow pumps. Vertical pumps are the most common style pump currently being used in flood control stations. Typically the slow rotative speed of the pump requires a reduction gear transmission to reduce the shaft speed of the diesel driver. Therefore, the general arrangement is a vertical axial flow pump driven by a diesel engine through a right angle speed reducer. For smaller horsepower irrigation and drainage applications, horsepower (Hp) requirements under 150 Hp, the axial flow pump is typically driven by a v-belt transmission and an electric motor. Vertical axial flow pumps have their impeller submerged and therefore are self-priming. For large pump units with an “over the embankment discharge arrangement” and a siphon assisted delivery, vacuum systems are required to minimize the engine horsepower for start-up. Prior too removal of all air from the piping system, the pump must provide the head to raise the water up and over the siphon crest at a minimum flow and critical depth. For small to medium pump capacities the additional head generally encountered for start up conditions and the size of the pumping units do not justify the cost of a vacuum system for reduction in the driver’s required horsepower. Therefore the pump and driver are selected for the maximum static head conditions for start-up. The additional expense and risk for automation or remote operation of the system is further justification for a self-priming pump.

For vertical style pump applications, the discharge pipe may be above or below the base plate, given there is a base plate. The below-base plate discharge style pump is the much more frequently used arrangement. The above the base plate discharge results in the driver at an elevation far above the operating level at the base plate and an elevated pump house floor. Site conditions may result in this arrangement being the best option. It does have an advantage in regard to the ease of disconnecting the pump from the discharge pipe. However, the above the base plate discharge is typically not used in the low head conditions of flood control pumping in south Florida because of the increase in head that is the result of the higher discharge pipe invert elevation.

Given no site constraints, it is often advisable to investigate the use of horizontal axial flow pumps for possible reduction of the height of the operating floor, reduction of the pump house square footage and the possibility of less system losses. Horizontal pumps have been utilized in many large capacity stations, some with pump propeller diameters greater than 120 inches. Some designers and operators prefer this style pump for the large capacity applications because of the better maintenance access to the drive shaft, bearings, and propeller. Horizontal pumps will require a vacuum priming system since the propeller is typically not submerged. They will also require a parallel shaft style reduction gear with the shaft in a horizontal position.

Rotodynamic pumps achieve their best efficiency at only one rate of flow and head, the BEP. Pumps can operate satisfactorily within a hydraulic range to the left (low flow) and to the right (high flow) of BEP. For large stations with frequent usage, higher pump efficiencies can lower the installed horsepower requirements and reduce capital and operating costs significantly. Therefore, the station designer should consider higher efficiency equipment on a life cycle basis. Energy consumption can be reduced if the following factors are considered when specifying the pump:

- It may be desirable to select a pump that has a lower peak efficiency but a flatter efficiency curve to address the static head range
- Engine: The efficiency of an engine is affected by the load. An engine will typically have a greater fuel consumption rate under part load
- An objective of the pump selection and the station design is to achieve the lowest total head and highest efficiency to provide the smallest driver and the lowest energy cost

Pumps have a preferred operating region (POR), for head and capacity. Operation of the pump within this region will not significantly affect the service life of the pump by the additional hydraulic loading, vibration, and flow separation. The POR for most centrifugal pumps is between 70 percent and 120 percent of the BEP. For high specific speed pumps the POR is between 80 percent and 115 percent of BEP. The allowable operating region (AOR) is the range of rates of flow and head over which the service life of the pump is not seriously compromised. Vibration levels exceeding the allowable limits is one criteria used by the pump manufacturers in establishing the AOR. Net positive suction head available (NPSHA), may also limit the AOR when the pump operation is over a wide range of flows.

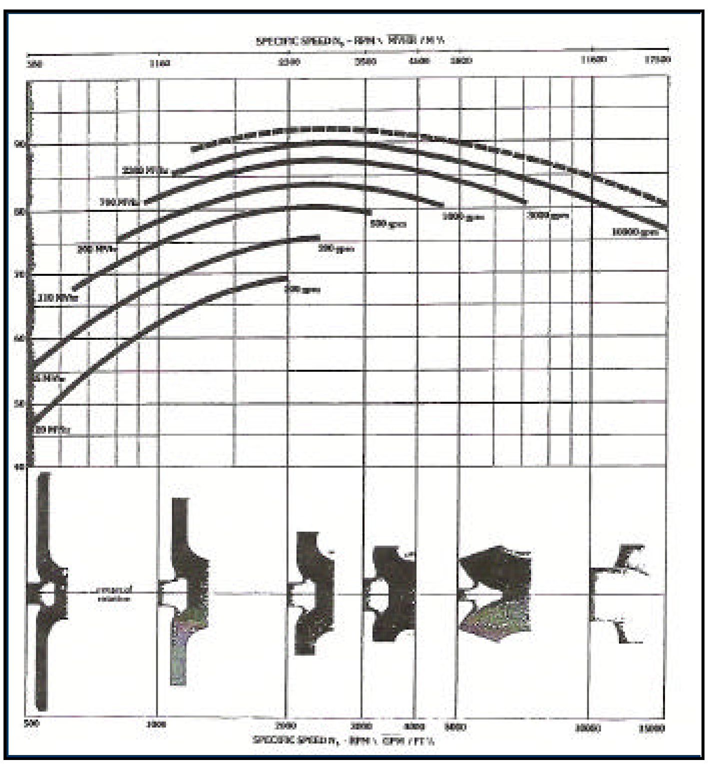
There are a number of additional pump design considerations that become a concern with operation at the limits of the AOR including:

- Flow velocities in the casing throat
- Stress limits of the shaft material
- Shaft fatigue
- Horsepower limitations
- Upthrust
- Suction recirculation
- Temperature rise
- Flow separation

The objective of the station designer as well as the pump designer is to ensure the POR satisfies the range of system heads. Operation outside this region should be limited. This objective is not just from the perspective of the service life of the mechanical components but also in regard to efficiency. Energy costs for pumps that see continuous use become the dominant factor in the life cycle cost analysis (LCC). Operation left or right of BEP can result in significantly lower efficiencies and therefore much higher energy use. A pump is normally selected on the basis of a duty point, i.e. a required head and flow or a working range of flow and head conditions. The pump designer selects a pump model and speed that satisfies the system requirements and establishes the BEP.

There is a general relationship between specific speed and efficiency. The effect of pump size (capacity) is also significant. See Figure 13.3-4

Figure 13.3-3 Relationship of Specific Speed to Flow and Efficiency



From this generalized illustration it can be concluded that for a high capacity application, the lower the specific speed of the pump the better opportunity for better efficiency. However, for a given duty point, a higher specific speed selection is a smaller pump at a higher speed, saving in both pump and driver cost. These factors must be weighed for the most cost effective and reliable design. The efficiencies of the figure represent the pump bowl efficiency which excludes losses within the pump components. The pump efficiency, the ratio of the pump output power to the pump input power that is the ratio of the water horsepower to the brake horsepower is somewhat less. Published data on pump efficiencies is not readily available from many manufacturers. Therefore bowl efficiencies shall be used in the analysis. In addition, since it is too premature to determine accurate efficiency maximums and minimums for the full operating range of the pumps, the efficiency at the rated condition shall be used in the energy calculations of the analysis.

The overall efficiency, the ratio of the energy imparted to the liquid by the pump to the energy supplied to the driver; that is the ratio of the water horsepower to the power input to the primary driver was estimated using 78 percent efficiency for the engine driver through the reduction gear. See the latter discussion in the LCC which attempts to estimate part load fuel consumption rate for the engine models selected for this analysis.

13.3.5.6 Pump Equipment General Arrangement

With the pump type established, station concept design alternatives can be developed that include appropriate intake and discharge structures that satisfy the hydraulic requirements as well as address the constraints and challenges of the site. To help with the development of the station's mechanical arrangement it is important to establish the operation and maintenance objectives including:

- Reliable facility capable of operation during extreme storm events to maintain flood protection
- Efficient operation to minimize energy costs
- Robust structural and mechanical systems to provide a long service life
- Flexibility to adjust operation to possible ultimate changed hydraulic conditions
- Automated operation to reduce operating costs
- Minimize maintenance costs by use of appropriate corrosive resistant materials and coating systems and the proper site and building design

The selected concept designs are typically compared using an LCC to establish the best cost alternative. With selection of the station design and its mechanical arrangement the requirements and characteristics of the system can be detailed and a final selection of the pump model can be selected. Iterative modifications of the station and/or system design may be required to achieve an optimum design. Such changes may be as dramatic as the number of pumps or pump type such as double suction vs. single suction to better match horsepower requirements to the available drivers.

13.3.5.7 Intake Alternatives

The intake's primary function is to provide the required flow to the pump. This flow should be uniform with a minimum of rotational wakes, free of harmful debris, have enough depth to prevent the formation of free surface vortices, and have an approach speed that avoids flow separation at boundaries. The pump intake also performs a number of other functions including providing the foundation for the service bridge and the pump house. The design objective from a cost perspective is to keep the depth and footprint size of this substructure to the minimum necessary to satisfy the hydraulic criteria. Therefore the structural design for the operating platform functions should accommodate the intake's minimum hydraulic geometric design in lieu of enlarging the intake to address these activities. However, in the case of rectangular intakes, from a hydraulic perspective, longer is better. So, if there is a need to increase the size of the intake, increasing its length will improve the flow characteristics to the pump.

The intake design is typically the responsibility of the pump purchaser. There are only a few available guidelines for the design of the pump intake, the more notable are the Hydraulic Institute Standard ANSI/HI 9.8-1998-Pump Intake Design, the British Hydraulics Research Association (BHRA) publication by Prosser, 1977 , U.S. Army Corps of Engineers Technical Letter Report by Triplett -1988 and the design guide recommended by Ingersoll-Dresser Pump Company -1991. The USACE Engineering Manual EM 1110-2-3105 also provides sump design guidance. The standard generally used in North America to design the geometry of the intake is the ANSI/HI 9.8-1998 - Pump Intake Design Standard. In accordance with section 9.8 of the hydraulic institute (HI) standard, each pump shall be installed in its own bay. The pump bay shall have adequate width and depth to limit the approach velocity to 1.5 fps.

The intake structure should be designed to allow the pumps to achieve their optimum hydraulic performance for all operating conditions. A good design ensures that the adverse flow phenomena are within the limits outlined in ANSI/HI Section 9.8.5.6. The hydraulic conditions that can adversely affect the pump performance and should not be present to an excessive degree are:

- Submerged vortices: Submerged vortices cause rapid changes in the local pressure on the pump propeller as a vortex core is ingested. This will result in severe vibration and cavitation.
- Free-surface vortices: Similarly, free surface vortices will cause rapid changes in the local pressure on the pump propeller as the vortex core is ingested, resulting in reduction of the pump discharge and a loss of efficiency. This, and any other air ingestion can cause fluctuations of impeller load which result in noise and vibration which may lead to physical damage.
- Swirl of flow entering the pump: Swirl is the ratio of the rotational and axial velocity components in the pump column. Swirl exists in the flow entering the pump if the tangential component of velocity is present in addition to the axial component. This condition does not comply with design assumptions in regard to the inlet velocity vector at the propeller vanes. Therefore, swirl in the pump intake can cause a significant change in the operating conditions for a pump resulting in changes in the flow capacity, power requirements and efficiency. It can also result in local vortex-type pressure reductions that induce air cores extending into the pump. Severe

swirling flow or pre-rotation, when centered on the pump axis has an additional effect on the pump performance that can either enhance or reduce the pump's performance depending on the direction of the rotation.

- Non-uniform spatial distribution of velocity at the impeller eye: Unsteady flow causes the load on the impeller to fluctuate, which can lead to noise, vibration and bearing problems.
- Entrained air or gas bubbles: Air gulping or aerated flow will reduce pump discharge and loss of efficiency. Small quantities of air can result in significant efficiency drop, i.e. 3 percent free air showed a drop of 15 percent efficiency.

Ideally, the flow of water into any pump should be uniform, steady, and free from swirl and entrained air. Lack of uniformity can cause the pump to operate away from the optimum design condition, and at a lower hydraulic efficiency. The negative impact of each of these phenomena on pump performance depends on pump specific speed and size, as well as other design features of the pump that are specific to a given pump manufacturer. In general, large axial flow pumps (high specific speed) are more sensitive to adverse flow phenomena than small pumps or radial flow pumps (low specific speed). A more quantitative assessment of which pump types may be expected to withstand a given level of adverse phenomena with no ill effects has not been performed.

- In designing an intake structure, the following points must be considered:
  - Flow from the forebay should be directed toward the pump inlets in such a way that the flow reaches the inlets with a minimum of swirl.
  - In order to prevent the formation of air-entraining surface vortices in the sump, the walls must be designed to avoid stagnation regions in the flow. A properly placed wall close to the inlet can reduce the tendency toward localized swirl and vorticity. The liquid depth also must be great enough to suppress surface vortices.
  - Although excessive turbulence or large eddies should be avoided, some turbulence does help to prevent the formation and growth of vortices.
  - The sump should be as small and as simple as feasible to minimize construction costs. However, the required sump volume may be specified for other reasons, such as to provide for a minimum or maximum retention time.
  - Large scale turbulence within the intake will cause uneven blade loading resulting in pump vibration and noise.
  - A distorted velocity profile caused for example by a clogged trash rack will promote swirl and vortex formation.
  - Boundaries between stagnant regions and the main flow tend to be unstable and fluctuate on position. These regions promote unsteadiness in the main flow and increase the chances of the formation of air entraining vortices.

For flood control and water supply stations of large capacity there are two basic intake types: Rectangular Intakes and Formed Suction Intakes.

Rectangular intakes are the most common intake with the water entering from the forebay of the structure into the intake and then the bell of the pump. The intake dimensions are determined as multiples of the pump bell diameter. The inlet bell diameter is based on limiting the bell inlet velocity to 5.5 fps.

Where:
A = Bell Area
Q = Pump Capacity
D = Pump Diameter
S = Submergences

$$\text{Bell Area (feet}^2\text{) } A = \text{Pump Capacity } Q / 5.5 \text{ fps.}$$

$$\text{Bell Diameter (feet) } D = 2\ (A / 3.14)^{0.5}$$

In accordance with the HI standard Section 9.7, the required minimum submergence to ensure there is no formation of free surface vortices is determined by the following empirical formula:

$$\text{Submergence } S = D + 0.58Q / D^{1.5}$$

The intake geometry recommended by the HI standard is based years of observation of constructed stations and numerous model studies. Table 13.3-1 summarizes the intake dimensions in accordance with the HI standards.

Table 13.3-1 Intake Dimensions per HI Standards

Description	Recommended Dimension of Bell Diameter
Bell centerline to entrance of intake and location of trash rack	5D[1]
Bell centerline to backwall	0.75D
Bottom of bell to floor	0.52
Minimum water depth above floor	S+0.5D
Bay width	2D
[1]D = Bell Diameter	

Formed Suction Intake (FSI) designs were developed by the USACE by the Hydraulics Laboratory in Vicksburg. The intake was developed to minimize the submergence needed as well as the width in comparison to the rectangular intake. The USACE experimented with a number of intakes and evaluated their performance based on the velocity distribution at the impeller. The geometry for the intake is presented in the USACE ETL 1110-2-327 as well as in Appendix I of EM 1110-2-3105. HI 9.8 also provides the geometry of the type 10 intake. The geometry is presented in terms of the throat diameter which commonly taken as the impeller diameter. The use of a FSI is typically limited to the large capacity pumps, 84-inch diameter and larger.

There is a significant difference in the submergence as determined by the USACE EM 1110-2-3105 and HI 9.8 with the HI intake being considerably deeper. Therefore, the use of the USACE method will significantly reduce pump station excavation, dewatering and substructure costs.

The use of the USACE method for determining submergence should be used with caution since there have been a number of physical model tests performed for various projects where the intake developed unfavorable flow conditions. Therefore for this project HI 9.8 shall be followed for the submergence calculations. During the physical model tests reduction in submergence shall be explored. At this point in the design, following Hydraulic Institute Standard 9.8 standards, there is very little difference between the calculated submergence of the two intake types. Subsequently, there are also only a few feet of difference in intake channel floor elevation between the two types of intakes. The FSI, however, can provide a significant difference in the width and length of the intake. Given the operating floor space requirements above do not govern the substructure size, (they typically do), there can being a footprint size reduction using the FSI.

It should also be noted, the ITT Industries has conducted model testing for their wet column mixed-flow pumps with both suction bell and the USCOE Type 10 FSI intakes. This testing was completed for the Davis Pond Diversion Project in St. Charles Parish, Louisiana. The rating for the prototype pump was 190 cfs at 16.6 TDH. The testing results revealed peak efficiencies with the FSI to be 81 percent; the same prototype tested with a suction bell intake configuration had approximately 87 percent peak efficiencies. ITT Industries attributes this difference to the FSI either due to additional head loss and/or because of a poorer flow pattern into the impeller.

The intake shall be provided a means to be dewatered for inspection, maintenance, and/or removal of the pumps. For smaller stations with an intake height from floor to the bridge of between 14 and 18+ feet the intake can be designed to be dewatered in a similar manner as the SFWMD’s spillways, with a needle beam and dewatering needles. The standard dewatering needles are aluminum interlocking structural frames with a height of 22 feet and widths of 2, 3, and 4 feet. The bottom of the needles are placed on a sill provided in the base slab and are leaned against the needle beam at a 10:1 angle. The needle beam may be a permanent installation or may be placed in vertical slots in the abutments or piers at the time of dewatering. For larger stations with deep intakes, dewatering bulkheads are used. These are structural steel gate like bulkheads that are lowered into slots in the abutments and piers by a crane. The bulkheads span the width of the bay and have equal heights so when stacked the top elevation is above the maximum suction design stage.

13.3.5.8 Discharge Works Alternatives

There are a number of discharge arrangement variations which can generally be categorized into two different styles: “over the embankment” or “through the embankment.”

13.3.5.8.1 Over-the-Embankment Discharge

For this discharge arrangement the invert of the discharge line or tunnel shall be equal to or be above the maximum discharge pool elevation to prevent backflow through the pumps when they are not running. This arrangement often includes the use of a siphon which can be justified on the energy saved due to the lower head when primed. The discharge line requires a relief/vacuum valve at the highest point to allow the venting of the air and closure when the siphon is established. The pump should be selected to operate over the entire range of heads provided by the siphon.

13.3.5.8.2 Through-the-Embankment Discharge

For this arrangement the discharge line runs horizontally through the embankment typically at an invert elevation that is established by the vertical height requirements of the pump. If the crest of the discharge pipe in always below the discharge pool elevation, this alternative will have no disadvantage in regard to energy usage relative to that of a siphon assisted discharge. However, this style discharge does require a means to prevent backflow through the pump when shut off. USACE EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations requires two means to prevent backflow. Typically the primary method for smaller discharge pipes is the use of a flap valve. For larger diameter pipes and discharge tunnels a motor operated slide or roller gate is specified. The second means of backflow prevention is a matter of interpretation of the USACE manual. Dewatering bulkheads are generally considered as an adequate backup for primary backflow prevention method. However, the USACE preference is to provide a second gate. The alternatives of this analysis, with the "through the embankment" discharge arrangements and flap gates, (Alternatives 3 and 4), have a second gate. This gate, however, is located in the intake and provides a means for dewatering of the pump suction bay of inspection and/or removal.

The top lip or crest elevation of the exit opening of the discharge tunnel or pipe for those alternatives with a vacuum priming system and assisted siphon flow delivery will require a minimum submergence below the predicted low groundwater elevation of the discharge channel. This low water elevation was assumed to be +5.6 feet NAVD88. The top lip or crest was conservatively set well below this low water stage to ensure a vacuum in the tunnel/pipe can be obtained as well as ensure the development of the siphon.

13.3.5.9 Description of Alternatives

As discussed, there is some degree of uncertainty in regard operational criteria of the station. The historic hydrologic flows would indicate the need for possibly small capacity (less than 1,000 cfs) flow capabilities. However, the future build-out of the EAA Reservoir A-1/STA-3/4 system has yet to be determined. For the purposes of this BODR, four arrangement alternatives were evaluated to determine the best arrangement for the new Pump station. Alternatives considered are discussed in Appendix 13-2, Mechanical Layout Alternatives Evaluation Technical Memorandum. Because the evaluation was conducted prior to the final sizing of the pump station, an estimated capacity of 3,000 cfs was used. The alternatives considered included:

- Three vertical axial flow pumps, each with 1,000 cfs capacity.
- Three horizontal axial flow pumps, each with 1,000 cfs capacity.
- Four vertical axial flow pumps, each with a 750 cfs capacity
- Five vertical axial flow pumps, each with a 600 cfs capacity

In regard to intakes, a FSI was selected for the larger pump alternatives, Alternatives 1, 2, and 3. The FSI will provide significant substructure width reduction in comparison with a rectangular intake while satisfying the operating floor space requirements, i.e. operating distance between engine drivers. The station substructure length is largely governed by the operating floor requirements, namely the service bridge, equipment lay down floor area within the pump house, control room, driver and reduction gear, and the trash rack and collection system. For the five pump alternative, Alternative 4, the rectangular intake's greater width affords additional space between the engine units, while the FSI begins to make the floor a bit too congested. Including the rectangular intake in the analysis also affords an opportunity to compare construction costs, admittedly not an apple to apples comparison, but possibly an indicator of the advantage of the rectangular intake's more inexpensive reinforced concrete unit costs.

The horizontal pump arrangement, obviously results in a number of uniquely different operating floor requirements. With the pump unit outside the pump house the area and height requirements of this facility are significantly reduced. However, the location of the control room must be located on one side of the house or the other. Since all the control rooms are elevated well above the operating floor level, location and orientation of the engine control panels to be within site of the operator should not be a problem. A standard USACE Type 10 FSI was used for the horizontal pump. The SFWMD's "Major Pump Station Engineering Guidelines" includes a sectional view of a very long formed intake with no defined geometry provided. This intake is apparently the proprietary design of the authors of the guideline so it was decided to use the USACE standard FSI. This results in an elbow prior to the pump. Typically a distance of (5) diameters from an elbow to the impeller is desired by the manufacturer to ensure a uniform flow to the impeller. This 50 feet length was considered excessive, 30 feet was provided, with the thought that flow straightening vanes could be provided in the elbow to help provide a uniform flow pattern.

Alternative 1 utilized a similar "up and over the embankment" discharge arrangement as its G-370 pump station with a discharge tunnel that gradually increased in cross sectional area and exits below low water so a siphon assisted delivery is ensured. The discharge tunnel design followed the SFWMD's "Major Pump Station Engineering Guidelines" and the "Hydraulic Design Criteria" for the inflow pump stations of STA-3/4, prepared by Burns & McDonnell, dated April 2000 in regard to target velocities, namely six fps at the tunnel crest and between two to four (3.3) fps at the discharge opening. The top lip of the tunnel opening was set well below minimum low groundwater stage. The width of the tunnel discharge opening is set at the same width as the intake bay.

Alternative 2 with the three 1,000 cfs horizontal pump utilized a "through the embankment" discharge arrangement. However, because of the large diameter of the pump, the pipe crest is above the maximum pool elevation. Therefore a discharge tunnel was added similar to Alternative 1, with its opening below low groundwater stage to again permit a siphon assisted delivery.

The last two alternatives, Alternatives 3 and 4 utilized a "through the embankment" arrangement with a straight horizontal discharge and backflow gates.

13.3.5.9.1 Alternative 1

This alternative shall have three vertical axial flow pumps with a rated capacity of 1,000 cfs (448,833 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the USACE Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the HI-9.8-1998, Pump Intake Design Standard. The discharge arrangement shall be an "up and over the protection elevation" arrangement with the discharge being pumped over a weir crest to a discharge tunnel. The exit of discharge tunnel shall be submerged to permit siphon assisted delivery of the flow as well as partial recovery of the velocity head. A vacuum system shall be provided to assist in the development of the siphon and removal of air from the discharge tunnel. The pump shall be driven by a diesel engine driver through a right angle reduction gear.

13.3.5.9.2 Alternative 2

This alternative shall have three horizontal axial flow pumps with a rated capacity of 1,000 cfs (448,833 gpm) each. The intake shall be a formed suction intake (FSI) in accordance with the geometry defined in the USACE of Engineers Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the Hydraulic Institute (HI) HI-9.8-1998, Pump Intake Design Standard. The discharge arrangement shall be a “through the embankment” arrangement with the discharge flowing horizontally from the pump to a discharge tunnel. Gates shall required to prevent backflow. The exit of discharge tunnel shall be submerged to permit siphon assisted delivery of the flow as well as partial recovery of the velocity head. A vacuum system shall be provided to prime the horizontal pump. The pump shall be driven by a diesel engine driver through a parallel shaft reduction gear.

13.3.5.9.3 Alternative 3

This alternative shall have four vertical axial flow pumps with a rated capacity of 750 cfs (336,625 gpm) each. The intake shall be a FSI in accordance with the geometry defined in the USACE Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the HI-9.8-1998, Pump Intake Design Standard. The discharge arrangement shall be a “through the embankment” arrangement with the discharge being pumped horizontally to the EAA Reservoir A-1 with use of gates to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear. *(This alternative was later found to be the least energy efficient and was not reviewed further.)*

13.3.5.9.4 Alternative 3A

After development of the system losses it was determined Alternative 3 be revised to include a turned down discharge and use of a siphon assisted flow delivery to make it more competitive with the other alternatives in regard to energy usage. This alternative shall have four vertical axial flow pumps with a rated capacity of 750 cfs (336,625 gpm) each. The intake shall be a FSI in accordance with the geometry defined in the USACE Engineering Manual EM 1110-2-3105. The submergence requirements shall be in accordance with the HI-9.8-1998, Pump Intake Design Standard. The discharge arrangement shall be a “through the embankment” arrangement with the discharge turned down with its exit submerged to take advantage of a siphon assisted delivery. Auto controlled slide gates shall be used to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear.

13.3.5.9.5 Alternative 4 and 4A

This alternative shall have five vertical axial flow pumps with a rated capacity of 600 cfs (269,300 gpm) each. The intake shall be rectangular intake in accordance with the geometry defined in the HI ANSI/HI-9.8-1998, Pump Intake Design Standard. The discharge arrangement shall be an “through the embankment” arrangement with the discharge being pumped horizontally to the EAA Reservoir A-1 with use of gates to prevent backflow. The pump shall be driven by a diesel engine driver through a right angle reduction gear. *(Alternative 4A was developed to eliminate the flap gate of the original Alternative 4 design and replace it with a* *backflow electric operated slide gate to reduce energy losses. Alternative 4 was dropped from further review.)*

13.3.5.10 Intake Geometry for Proposed Alternatives

The formed suction intake geometry for the proposed alternative flow rate is indicated in Table 13.3-2. The rectangular intake geometry for the proposed alternative flow rate is indicated in Table 13.3-3.

Table 13.3-2 Formed Suction Intake Geometry

Geometry	Alternative 1	Alternative 2	Alternative 3A
Capacity at rated condition (cfs/gpm)	1,000/448,830	1,000/448,830	750/336,623
Throat Diameter (inches/feet)	120/10	120/10	102/8.5
Submergence (inches/feet)	290/24.2	290/24/2	257/21.4
Pump centerline to entrance (feet)	33.0	33.0	28.05
FSI height (feet)	8.80	8.80	7.48
FSI width (feet)	23.10	23.10	19.64
Min. water depth above floor (feet)	28.57	28.57	25.15
*Low Water shut-off elevation (feet NAVD88)	5.6	5.6	5.6
Intake slab elevation (feet NAVD88)	-20.71	-20.71	-16.75

*assumes one foot head differential across trash rack due to blockage at design low water

Table 13.3-3 Rectangular Intake Geometry

Geometry	Alternative 4A
Capacity at rated condition (cfs/gpm)	600/269,298
Bell Diameter (inches/feet)	11.79/109
Submergence (inches/feet)	234/19.5
Bell centerline to entrance (feet)	58.9
Bell centerline to backwall (feet)	8.8
Bell inlet to floor (feet)	5.9
Bay width (feet)	23.6
Min. water depth above floor (feet)	25.41
*Low Water shut-off elevation (feet NAVD88)	5.6
Intake slab elevation (feet NAVD88)	-17.01

*assumes one foot head differential across trash rack due to blockage at design low water

13.3.6 Mechanical Engineering Analysis and Design

13.3.6.1 General Mechanical Arrangement Design for Station Alternatives

13.3.6.1.1 General Pump Station Description

The mechanical design of the pump station includes the pumps, pump drivers, and appurtenances. The alternative conceptual designs of the pump station all differ in the mechanical arrangement, and the subsequent power requirements. All alternatives of the pump station shall utilize axial flow pumps with gear reduction transmissions, and diesel engine drivers. The pump station equipment layout shall generally be in accordance with USACE EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations and the SFWMD's "Major Pump Station Engineering Guidelines". The alternatives shall include all required and preferred auxiliaries including, flexible connections, tension bars, backflow prevention, vacuum priming systems and air vents, lube oil systems, waste lube oil system, etc. as appropriate. The pump units will be capable of local/manual, local/auto and remote operation. Stilling wells with water level sensors and low water level shut off switches shall be installed in each pump bay. The diesel engine drivers shall be water cooled. A diesel fuel system shall include aboveground double-walled fuel storage tanks, double walled fuel piping, leak monitoring devices, fuel level measurement, filling pipes, venting pipes, access manholes, etc. The fuel storage system will be designed to hold ten days of fuel for continuous operation of all pumps and accessories. The pump house shall have ventilation as required for the equipment operation. The supply fans for building ventilation shall provide filtered air and shall be thermostatically controlled. An air-conditioned control room, as well as break room with restrooms shall be provided. An air-conditioned electrical room shall be provided for the electrical control equipment. A standby power engine-generator w/ back-up shall be provided for the building service loads. Trash racks with motorized rakes shall be provided to protect the pumps. The trash racks shall be designed to minimize head loss and protect the pump in accordance with the pump manufacturer's recommendations. The rack and rake shall be designed for easy removal and maintenance. All subaqueous components are to be corrosive resistant. The rake shall operate when the pump is operating. Trash shall be deposited in a dump site. The rake shall have a back-up redundant system.

13.3.7 System Analysis of Station Alternatives

13.3.7.1 System Design Requirements

The USACE Manual EM 1110-2-3105, Mechanical and Electrical Design of Pump Stations provides a good reference for the system design of the station and development of the operating range for the pump including the static heads, friction losses, exit losses and the velocity head. In the design phase, the engineer develops station system curves for the operating range established by the hydrologic design. This operating range consists of the maximum and minimum intake and discharge stages and the corresponding flow capacities. This analysis typically results in two station system curves, one curve representing the design flood condition or a low head/high capacity condition, and the other representing a maximum operating condition or a high head/low capacity condition. The intersection of the pump curve with these system curves represents the expected performance range for the pump. Since the system curves are based on hydrological probabilities, the design engineer often adds a conservative margin to the performance requirements of the pump to ensure there is adequate horsepower and capacity to address the unexpected event.

13.3.7.2 System Analysis

Based on the "Hydraulic Design Criteria" for the inflow pump stations of STA-3/4, prepared by Burns & McDonnell, dated April 2000, the following pumping conditions for all of the alternatives shall be assumed as presented in Table 13.3-4. The static head conditions for the alternatives are as presented in Table 13.3-5.

Table 13.3-4 Pumping Conditions for all of the Alternatives

Maximum Suction Stage	Normal Suction Stage	Low Water Stage	Low Water Shut-off	Maximum Discharge Stage	Minimum Discharge Stage
14.0 feet	8.6 feet	6.6 feet	*5.6 feet	20.6 feet	8.6 feet

*Based on a one head differential across the trash rack at low water stage.

Table 13.3-5 Pumping Static Head Conditions

Alter-native	Maximum Static Head (feet) w/siphon	Maximum Static Head (feet) w/o siphon	Minimum Static Head (feet) w/siphon	Minimum Static Head (feet) w/o siphon	Normal Static Head (feet) w/siphon	Normal Static Head (feet) w/o siphon	Start-up Head (feet) w/ suction at 8.6
1	13.6	21.6	-5.4	14.6	-1.4 to 10.6	18.6	*14.6
2	13.6	16.6	-5.4	9.6	-1.4 to 10.6	13.6	13.6
3	N/A	13.85	N/A	6.85	N/A	10.85	10.85
3A	13.6	13.85	-5.4	6.85	-1.4 to 10.6	10.85	10.85
4A	N/A	13.6	N/A	0.35	N/A	4.35	4.35

All elevations in accordance with NAVD88
*Based on SFWMD design guide the pump must be able to pump to the mid-height of the discharge tunnel above the weir crest.

For the purpose of the mechanical system design the static heads are calculated from the elevation of the water in the intake on the pump side of the trash rack. In addition to the static head there are the dynamic losses that include:

- Friction loss in the pipe
- Loss in the pipe bends
- Loss at the outlet
- Loss at the bell inlet or FSI inlet
- Flap valve loss

For this analysis the total head for the rated pump capacity shall be established as an initial step towards defining the BEP for the pump. For the purposes of determination of system losses, and as a safety margin EM 1110-2-3105 recommends the entire velocity head be considered unrecoverable and thereby added to the other losses. The discharge tunnels for Alternatives 1 and 2 have a gradually expanding geometry to slow velocities to between two and four fps at the exit. Since there will be some velocity recovery with the tunnel design the exit loss shall be ignored for these alternatives.

Losses at the entrance, bends, and valves are called minor losses as distinguished from the friction loss in the straight pipe. All energy loss terms can be expressed in terms of the local average velocities.

$$h = K (V^2)/2g$$

Where: h = head
K = coefficient
V = Velocity
G = Gravity

The coefficient "K" is an empirical value developed from experiment. There are a number of references that can be used to find the value of "K" for various system components i.e. "Handbook of Hydraulics" Brater and King and "Cameron Hydraulic Data" produced by Ingersoll-Rand Company.

	FSI Entrance	Bell Entrance	45° Elbow	90° Elbow	Flap Valve
K	0.15	0.05	0.18	0.25	1.0

To develop a feel for the system's dynamic losses at the rated capacity for comparative purposes Table 13.3-6 was prepared.

Table 13.3-6 System Dynamic Losses at Rated Capacity

As Feet of Head Loss								
Alter-native	**Capacity (cfs)**	**Velocity (fps)**	**FSI Entrance**	**Bell Entrance**	**45° Elbow**	**90° Elbow**	**Flap Valve**	**Total**
1	1,000	12.74	0.38		0.45	0.63		1.46
2	1,000	11.93	0.33		0.40	0.55		1.28
3	750	13.22	0.40			0.67	2.71	3.78
3A	750	13.22	0.40		0.49	0.67		1.56
4	600	13.59		0.14		0.71	2.87	3.72
4A	600	13.59		0.14		0.71		*1.84
As Equivalent Feet of Straight Pipe Using the Cameron Data								
Alter-native	**Capacity (cfs)**	**Velocity (fps)**	**FSI Entrance**	**Bell Entrance**	**45° Elbow**	**90° Elbow**	**Flap Valve**	**Total**
1	1,000	80	75		75	100		330
2	1,000	50	75		75	100		300
3	750	25	75			100	350	550
3A	750	50	75		75	100		300
4	600	25		25		100	350	500
4A	600	25	25	100	150			
*1.0 ft. added for exit loss								

The losses in the portion of the pump that is supplied by the manufacturer, (between the suction bell and the end of the discharge elbow) are considered internal pump losses and are typically not included in the system head loss determination. The calculation of these internal pump losses is the responsibility the pump manufacturer. For this analysis to get a reasonable estimate of energy cost for the LCC, 2.5-feet of head loss has been added to the total head of all the alternatives for the internal pump loss. *From the review of this and the previous table modification to Alternatives 3 and 4 were made resulting in Alternatives 3A and 4A. Details are presented in a later discussion.*

13.3.7.3 Pump Performance Requirements

13.3.7.3.1 Rated Condition

The normal operation, or the most frequent operating condition, shall be for stage control during less than design storm events. Typically, running all pumps to address inflows from normal rain events or from the maintenance of the groundwater stages within the basin will result in a relatively quick drawn down of the NNRC given the agricultural inflow stations have ceased operation. The station operation, therefore, will be often restricted to one pump. As discussed, the number and capacity of pumps selected for this analysis to address the potential EAA Reservoir A-1 operational plan will provide a valuable comparative analysis of an array of pump combinations. However, the station's priority is to reliably operate during the standard project flood (SPF) conditions which potentially at the start of the event may be at the canal's minimum stage. In time the canal stages under these flood conditions will approach the maximum canal stage conditions and the pool to pool head differential would decrease. As discussed in the operational model, the EAA Reservoir A-1 stage will likely have seasonal schedules, with the wet season stage lower to provide more storage for the traditionally larger basin runoff. Consequently, the pumping operations during normal operating conditions as well as the flood events during this season could potentially have much lower head conditions. This is not reflected in the model to any significant degree since the suction stage was taken as elevation 8.6 NAVD88 for all pumping to simplify the analysis. The future design phases of the station will need to more accurately predict the seasonal stage conditions so the system curves will reflect the actual pump performance requirements. It should be emphasized, for low head flood control pumps, the static, pool to pool, head represents the primary portion of the total system head and therefore the over-riding and critical consideration in the establishment of the rated condition. This static head range could present a significant challenge to the manufacturer in regard to ensuring the operations are within the preferred operating region of the pump. The intersection points of the minimum static head system H/Q curve and the maximum (EAA Reservoir A-1 full) static head H/Q curve with the pump H/Q curve may be located in undesirable or unstable regions of the pump curve or at the minimum head a runout condition may occur that causes a net positive suction head (NPSH) problem. In addition, the more or less double duty point caused by the seasonal variation in operation will make it very unlikely there will be an optimum operating condition for the BEP for the pump. Dry season operation should be to the left of the BEP and during wet season with the stages in the EAA Reservoir A-1 lowered, the operating point moves to the right of BEP. Therefore, the BEP-rated condition point should be located to the left of the design flood (minimum static head condition) duty point to have the best efficiency of the pump at the normal operating conditions. In other words, the rated condition point should be located between the design flood/minimum pool stage (at 110 percent of the rated flow) and maximum EAA Reservoir A-1 stage (at the 80 percent of the rated flow) on the pump H/Q curve.

To attempt to define the pump's rated condition, the head at the rated capacity was set at or near the average head conditions established by our hypothetical operational model, EAA Reservoir A-1 stage 14.24 NAVD88 (wet season), and 16.15 NAVD88 (dry season) with a suction stage at the normal level of 8.6 NAVD88. For Alternatives 1, 2 and 3A with the siphon assisted delivery, this static head defines where their BEP should be. For Alternative 3, with its pipe crest at 20.85 NAVD88 then 12.25 should be its rated point. The rated capacity and head for the pump alternatives are summarized in Table 13.3-7.

Table 13.3-7 Pump Design Capacity at Rated Conditions

Alter-native	Capacity (cfs/gpm)	Velocity Head (feet)	Static Head (feet)	Friction Head (feet)	Pump Losses	Total Head (feet)	*Brake HP
1	1,000/448,833	2.5	7.5	2.8	2.5	15.3	2,221
2	1,000/448,833	2.2	7.5	2.3	2.5	14.5	2,104
3	750/336,626	2.7	12.25	4.6	2.5	22.1	2,405
3A	750/336,626	2.7	7.5	3.1	2.5	16.8**	1,836
4	600/269,300	2.9	7.75	5.0	2.5	18.1	1,582
4A	600/269,300	2.9	7.75	2.5	2.5	16.6**	1,450

*Assumes a system efficiency of 0.78; 80 percent for pump and 96 percent for gear

**1.0 foot added for the exit loss

The maximum head conditions for the alternatives shall be used to establish the engine's maximum horsepower rating and therefore permit the selection of an engine model. This maximum head condition was assumed to be at 80 percent of the rated flow condition. It should be noted as shown in Table 13.3-8 there is a significant difference in horsepower requirements for Alternative 1 if the vacuum system is not used to prime the siphon. For the horizontal alternative (Alternative 2), there is no choice but to use the vacuum system since it is needed to prime the pump.

Table 13.3-8 Pump Design Capacity at Maximum Static Head Conditions

Alter-native	Capacity (cfs/gpm)	Velocity Head (feet)	Static Head (feet)	Friction Head (feet)	Pump Losses	Total Head (feet)	*Brake Hp
1	800/359,066	1.6	16.0	1.8	2.5	21.9	2,548
**1	800/359,066	1.6	23.0	1.8	2.5	28.9	3,361
**2	800/359,066	1.4	18.0	1.5	2.5	23.4	2,719
3	600/269,300	1.7	15.3	3.0	2.5	22.5	1,964
**3A	600/269,300	1.7	15.0	2.0	2.5	22.3***	1,942
4	480/215,440	1.8	17.5	3.3	2.5	22.6	1,578
4A	480/215,440	1.8	17.5	1.6	2.5	22.0***	1,532

*Assumes a system efficiency of 0.78: 80 percent for pump and 96 percent for gear
** Start-up w/o vacuum assist
***1.0 foot. added for exit loss

As stated in the SFWMD's "Major Pump Station Engineering Guidelines" the engine horsepower shall be sufficient to pump water without vacuum assistance during low water suction conditions to a minimum of 50 percent of the discharge tunnel height at the crest at 50 percent of the rated flow capacity. It should be noted, for the arrangements with the siphon assisted delivery, the total head for the start-up condition equals the distance from the suction elevation to the top surface of the critical depth over the crest plus the frictional and minor losses to the siphon crest plus the velocity head at the critical depth and velocity.

$$TH = Z1 + hf\ (1 - s) + (V_c^2)/2g$$

The total head for the siphon assisted condition equals the difference between the suction and discharge elevations plus the frictional and minor losses for the system plus the velocity head at the discharge.

$$TH = Z2 + hf\ (1 - s) + (V_c^2)/2g$$

For the operational model the rated capacity was used in the calculation of losses as well as the same mechanical efficiencies for determination of the horsepower requirements since the pumps' rated conditions were established with the wet season and dry season average stage as a target for the pumps' BEP. Summaries as presented in Table 13.3-9 shall be used in the calculation of the average annual energy usage for the life cycle analysis.

Table 13.3-9 Pump Operating Conditions for Energy Usage Determination

Alter-native	Capacity (cfs/gpm)	Velocity Head (feet)	Static Head (feet)	Friction Head (feet)	Pump Losses	Total Head (feet)	*Brake Horse-power
Wet Season							
1	1000/448,833	2.5	5.64	2.8	2.5	13.5	1,951
2	1000/448,833	2.2	5.64	2.3	2.5	12.6	1,834
3	750/336,626	2.6	12.25	4.6	2.5	22.1	2,405
3A	750/336,626	2.7	5.64	3.1	2.5	15.0**	1,633
4	600/269,300	2.9	5.75	5.0	2.5	16.1	1,407
4A	600/269,300	2.9	5.75	2.5	2.5	14.6	1,275
Alter-native	**Capacity (cfs/gpm)**	**Velocity Head (feet)**	**Static Head (feet)**	**Friction Head (feet)**	**Pump Losses**	**Total Head (feet)**	***Brake Horse-power**
Dry Season							
1	1000/448,833	2.5	7.55	2.8	2.5	15.3	2,228
2	1000/448,833	2.2	7.55	2.3	2.5	14.5	2,112
3	750/336,626	2.7	12.25	4.6	2.5	22.1	2,405
3A	750/336,626	2.7	7.55	3.1	2.5	16.9**	1,841
4	600/269,300	2.9	7.55	5.0	2.5	18.9	1,651
4A	600/269,300	2.9	7.55	2.5	2.5	17.4**	1,520

*Assumes a system efficiency of 0.78: 80 percent for pump and 96 percent for gear
**1.0 foot added for exit loss

13.3.7.3.2 Discussion of System Analysis Results

It is evident from the system analysis presented above and as initially suspected Alternative 3 because of its high discharge pipe crest and straight horizontal discharge without assisted delivery of a siphon will have high annual energy costs. The savings of initial construction cost are considered not sufficient to make this a competitive alternative in the life cycle cost analysis. Therefore this alternative will not be considered further. Alternative 4 high friction head losses are for the most part due to the flap valve. It will make a significant energy reduction if this valve is replaced with a backflow gate that is opened automatically on pump start-up and closed automatically with the pump's shut down. Therefore Alternative 4A was developed and Alternative 4 shall be dropped from further consideration. Table 13.3-10 has been revised to reflect these changes.

Table 13.3-10 Remaining Alternative Operating Conditions for Energy Usage Determination

Alt.	Capacity (cfs/gpm)	Velocity Head (feet)	Static Head (feet)	Friction Head (feet)	Pump Losses	Total Head (feet)	*Brake Horse-power
Wet Season							
1	1000/448,833	2.5	5.64	2.8	2.5	13.5	1,951
2	1000/448,833	2.2	5.64	2.3	2.5	12.6	1,834
3A	750/336,626	2.7	5.64	3.1	2.5	15.0**	1,633
4A	600/269,300	2.9	5.75	2.5	2.5	14.6	1,275
Alt.	**Capacity (cfs/gpm)**	**Velocity Head (feet)**	**Static Head (feet)**	**Friction Head (feet)**	**Pump Losses**	**Total Head (feet)**	***Brake Horse-power**
Dry Season							
1	1000/448,833	2.5	7.55	2.8	2.5	15.3	2,228
2	1000/448,833	2.2	7.55	2.3	2.5	14.5	2,112
3A	750/336,626	2.7	7.55	3.1	2.5	16.9**	1,841
4A	600/269,300	2.9	7.55	2.5	2.5	17.4**	1,520

*Assumes a system efficiency of 0.78: 80 percent for pump and 96 percent for gear
**1.0 foot added for exit loss

13.3.7.3.3 Pump Characteristics and Performance Requirements

With the establishment of the operating conditions and pump capacity, the design engineer can establish an estimated propeller size, pump speed, efficiency, and required horsepower for the rated point by consulting manufacturer's catalog data sheets or by calculation. There are a number of pump design parameters that should be calculated to provide additional performance criteria for the eventual selection of a pump model. A pump index that classifies a pump in accordance with its characteristics is the parameter "specific speed". This index defines the optimum rotor geometry for the maximum efficiency of any size of pump. It is used as a guide for selection of an impeller type for a given operating range.

$$Ns = Nt\,(Q^{0.5})/H^{0.75}$$

Where:

Ns = pump specific speed

Nt = pump rotative speed, (rpm)

Q = flow at the BEP (gpm)

H = head at the BEP (feet)

This index number should not be confused with the "Suction Specific Speed" which is a similar index that describes the suction characteristics of the pump:

$$Nss = Nt\,(Q^{0.5})/(NPSHR)^{0.75}$$

Where:

Nss	= suction specific speed
Nt	= pump rotative speed, (rpm)
Q	= flow rate, runout condition (gpm)
NPSHR	= net positive suction head required (feet)

The suction specific speed can be used to determine the maximum permissible speed of the pump. The net positive suction head required, (NPSHR) is the suction condition required by the pump to operate without cavitation. The maximum rotative speed can be calculated using Nss = 8,500, a typical and conservative value for a pump operating at its BEP.

The rated capacity (Q) should be within the region of 80 to 110 percent of the BEP for the furnished propeller. The following table represents a very preliminary analysis of the pumps for the four alternatives and has been established to provide input for the life cycle cost analysis. Follow-up by review of the station requirements by two or more manufacturers will provide the designer more surety in his selection decision. However, the designer, if the job is to be bid competitively as is the case of public works projects, should avoid any preferential treatment that may cause a protest by the competing manufacturers. Therefore it is advisable to solicit a professional independent pump expert to assist in the development of the specifications of the pump. The selected alternative as presented in Table 13.3-11 will need further review and the input from such an expert to ensure the parameters are within the range of the capabilities of a manufacturer.

Table 13.3-11 Pump Specification Parameters

Pump Alternative	**QBEP** (cfs)	**S** (feet)	**Dimp** (inc.)	**HBEP** (feet)	**Maximum rpm**	**rpm BEP**	**Tip Speed** (fps)	**NPSHA** (feet)	**Ns**
1	1,000	24.2	120	12.0	208	153	80.15	46.9	14,987
2	1,000	24.2	123	12.0	137	130	69.79	26.9	15,499
3A	750	21.4	102	15.0	237	182	80.75	46.1	11,136
4A	600	19.5	90	15.0	265	204	80.19	46.2	13,910

Suction specific speed was assumed to be 8500
S= submergence as per HI 9.8

These requirements are enforced by the factory performance test in accordance with ANSI/HI 2.6-2,000 Vertical Pump Test standard. The test tolerances need to be specified. The engineer has a choice between the ANSI/HI test standard, ISO 115 - Hydraulic Performance Acceptance Test standard or an independent version of the engineer's own design. In accordance with the ANSI/HI standard acceptance criteria the rate of flow tolerance at rated pump total head and speed can vary from +10 percent to 0 percent of the specified Q. Efficiency shall be equal to or greater than the specified minimum efficiency at the rated flow, total pump head, and speed. The engineer should be aware of this ANSI/HI standard for the details of these acceptance criteria.

The pump input horsepower requirements shall not over-load the selected driver throughout the pump curve from shut off head to the maximum operation flow range. Minimum submergence shall not be less than the submergence at maximum design flow range.

There are a number of physical parameters the pump manufacturer needs to know relative to the station which are summarized below:

- Protection elevation
- Design low water (suction)
- Intake floor elevation
- Operating floor
- Design high water (discharge)
- Design high water (suction)
- Discharge Pipe Invert

The operating floor elevation should be set to ensure to equipment is above the projected flood stages as established by the hydrologic and hydraulic analysis. When the station is exposed to the discharge pool elevations, the operating floor elevations should be no lower than the top of the embankment. If the station is located on the inflow side then flood stages estimated by the hydrologic study will set the elevation of the operating floor. This elevation should be at a minimum four feet above the Project design flood stage.

There are a number of miscellaneous design requirements that need to be addressed. The pump and speed reducer need to be designed such that no damage will occur in the event of reverse rotation caused by back-flow. This is facilitated by locking the head shaft through employment of a backstop in the speed reducer. The force used in design of the non-reverse ratchet of the speed reducer is calculated using the highest discharge head and the lowest intake water elevation with a safety factor of 2.0. All pumps of the same type and designation need to be identical pumps as defined by ANSI/HI 2.1.6.3 – 2,000. This standard requires the pump to be a duplicate of the original pump as closely as manufacturing tolerances allow. Performance characteristics shall be the same within the allowable limits specified. Another requirement is the shaft rotation needs to match the typical rotation of the driver. The pump is custom designed, and therefore the more economical component of the system to be varied to comply with the standard model stock driver.

13.3.7.4 Model Studies

Physical modeling of large intakes and connector canals is generally considered a necessary stage in the design of large capacity pump stations. The HI standard ANSI/HI 9.8 - 1998 recommends all intakes of pump stations with a total station capacity over 100 cfs be model studied. However, the designer must decide the necessity of a model study on a case by case basis. The cost for smaller stations may be prohibitive. The need for model testing should be considered for:

- Multiple pumps with a common connector canal and a multitude of possible pump operating sequences
- Station capacities greater than 100 cfs
- Intakes with possible obstructions to the approach flow close to the pump location such as debris blockage of trash racks or screens

- Pump intakes with asymmetric approach flow and the submergence Froude number, $Fs = VD/(gS)^{0.5} > 0.25$
- Intakes with minimum water levels below recommended values

It is usually impossible to determine all the essential facts for a given fluid flow by pure theory. Fluid dynamics is more heavily involved with empirical work than is structural engineering, machine design, or electrical engineering because the analytical tools presently available are not capable of yielding exact solutions. The solutions of most engineering problems involving fluid mechanics rely on data acquired by experimental means. Therefore dependence must be placed upon experimental investigations. The number of test to be made can be greatly reduced by a systematic program based on dimensional analysis and the laws of similitude or similarity. These laws permit the application of certain relations by which the test data can be applied to other cases. The laws of similitude make it possible to predict the performance of the prototype from tests made with a model.

Geometric similarity means the model and the prototype are identical in shape but differ only in size. The scale factor or the ratio of the linear dimensions of the prototype to the corresponding dimensions of the model is an important consideration to ensure an accurate model.

If two systems are dynamically similar, corresponding forces must be in the same ratio. Dynamic similitude is achieved when two flow systems which a geometrically similar satisfy the dimensionless equation of motion. Any deviation is termed a scale effect. The dimensionless terms that must have the same value in both flow systems include:

- Relative submergence = h8 / ro
- Circulation number = Gn = Gro / Q
- Froude number $Fn = (Q / ro\ h8) / (g\ h8)^{0.5}$
- Reynolds number = Rn = Q/v h8

There is little reference material available on hydraulic model testing. HI 9.8.5 - Model Test of Intake Structures is a good educational section that provides the modeling basics.

The objective of a model study is to ensure the intake design generates favorable flow conditions in the inlet to the pump. Intake models are operated using Froude similarity since the flow process is controlled by gravity and inertial forces. In modeling an intake it is important to select a reasonably large geometric scale to minimize viscous and surface tension scale effects and reproduce the flow pattern in the vicinity of the pump. The model must be large enough to allow visual observations of the flow patterns, accurate measurements of swirl and velocity distribution and sufficient dimensional control.

Comparison of model to prototype in regard to vortex formation indicates negligible scale effects for Froude scaled models with weak vortices and surface dimples. Some scale effects were detected for models in which air core vortices occurred. Compensation for these scale effects is possible by some increase in model flow above the Froude scaled value. It is important the Reynolds and Weber numbers be sufficiently high to avoid the potential of scale effects. Models at higher scale ratios yield higher Reynolds and Weber numbers at the same Froude number.

13.3.7.5 Evaluation of Alternatives

13.3.7.5.1 Structural Design Considerations

13.3.7.5.1.1 Foundation

The assumption was made that a mat foundation would provide a more than adequate foundation for the northeast pump station. The greatest bearing loads from the dead load of the structure typically occur during the construction period when the structure is dry. However, there is the analytic consideration that the lower substructure will actually be flooded prior to construction of the structure above. Nevertheless, a preliminary review indicates loading should be well below allowable soil bearing pressure.

13.3.7.5.1.2 Stability Analysis

The stability of the northeast pump station is not a concern but the selected alternative should be analyzed to ensure it is well within the safety factors required by the engineering design standards.

13.3.7.5.1.3 Reinforced Concrete Design

There was only a preliminary analysis performed to size of the various abutment walls, piers and wingwalls. All structure alternatives will provide ample opportunity for lateral support of the abutments. The other structural components, thicknesses, depths, etc. were based on experience with the intent to provide a conservation estimate of the concrete cubic yards that will be required to construct the major reinforced concrete components of the structure. The dimensional adjustments that will be required by the future detailed design of the structure will not dramatically change the initial cost of the alternatives.

13.3.7.5.2 Pump House

A typical precast concrete panel with steel frame construction was assumed for proposed pump house alternatives. The pump house shall be required to be designed to satisfy SFWMD wind load standards that will substantially increase the panel and resisting frame sizes over that required by the Florida Building Code. Because of these wind load requirements precast double tees were used in the analysis. The future design stages should review the possible advantages of a metal roof joist and cast in place deck roof system. The building size varies in width with each alternative from 50 feet x 74 feet to 50 feet x 98 feet. The eave height varies with each alternative due to the clearance requirements for removal of the major equipment by the bridge crane. A four foot parapet was used in the analysis as is recommended due to the wind load requirements. The bridge crane height and the resulting eave height shall need further verification when the equipment dimensions are established.

13.3.7.5.3 Construction Considerations

Dewatering of the structure site to permit construction of the intake in the dry is a key construction task which will determine the success of the project. It was assumed a cofferdam will be required to dewater the site due to the relatively porous soil characteristics. The cofferdam tip elevations and top elevations were all assumptions that were applied equally to all alternatives therefore providing no advantage to any one alternative in this comparative analysis. It was assumed there would not be the need for tremie seal and the cofferdam could be dewatered from sumps by pumping. However, all of these assumptions must be analyzed during the next phase of the design of the station. This dewatering facility is envisioned as a steel sheet pile cofferdam of a foot print size just large enough to allow construction of the substructure. Internal bracing with a tie-back system would be necessary for structural support of the sheets.

13.3.7.5.4 Life Cycle Cost Analysis (LCC) Summary

The LCC is described and summarized in Appendix 13-2 Mechanical Layout Alternatives Evaluation Technical Memorandum. The capital costs for the four alternatives were about the same, with less than 1.5 percent difference between the least costly and the highest cost alternatives, essentially no difference considering the accuracy of conceptual level estimates.

The LCC included the cost of energy, operation, and maintenance as well as capital cost for the service life of the facility. Financial factors assumed for the analysis included:

- Inflation rate at 3.5 percent
- Interest rate at 6.0 percent
- Discount rate = interest rate - inflation rate = 2.5 percent
- Review period = 25 years
- Discount factor for 25 years. and 2.5 percent discount rate = 18.47
- Cp/Cn for 25 years and 2.5 percent discount factor = 0.58

The life cycle costs were also very similar, with about five percent difference between alternatives.

Given the relative minor difference in the present value totals for the alternatives and the margin of error in the estimating of not only the construction costs but also the potential pump performance at this early stage of the design, the LCC analysis does not provide a conclusive result as to the optimum station alternative. But there is an important finding that is evident from the analysis. For this station, which could see many more hours of continuous duty than the typical SFWMD flood control station, energy costs are extremely important. This is especially true considering the cost of fuel is escalating at a rate beyond that of the 3.5 percent inflation rate used in the analysis. Therefore the station design needs to incorporate every efficiency measure that is available to minimize system losses and reduce fuel consumption regardless of the initial cost. This will result in the optimum station design from a life cycle cost perspective.

Given that the initial costs for all the alternatives are in essence equal, we recommend a 600 cfs pump station. The general arrangement drawing is a suggested arrangement that makes use of the advantages of the various alternatives outlined above, and provides the greatest flexibility for operation of the current and future flood control and water supply system. This station design has the following:

- Self-priming with no vacuum system required reducing potential complications for remote operation of station
- The most number of pumps with the smallest capacity provides the most pumping flexibility
- With the most number of pumps, provides least impact to station operation when a pump unit is off line

- Low entrance losses due to bell intake
- Lowest pipe crest elevation requiring low start-up horsepower without vacuum priming
- Recovery of velocity head and exit losses due to discharge tunnel
- Steel fabricated discharge tunnel to reduce friction losses and lessen construction cost
- Lowest operating static heads due to siphon assisted delivery
- Low friction losses due slower flow velocities as a result of the use of larger impeller and slower rotative speed for this 600 cfs rated capacity
- Reduced height of substructure due to "through the embankment" discharge arrangement
- Control room and break rooms at opposite side of pump house from engines permitting optimum viewing of operating floor equipment and engine control panels
- Control room and break rooms at opposite side of pump house from engine and exhaust system reducing noise
- Engines located close to exterior wall permitting intake ventilators to be located in close proximity to engines for optimum ventilation arrangement
- Electric start engines eliminating need for large capacity compressed air system and potential complication for remote starting
- Reduced cost of substructure due to rectangular intake

13.3.8 Station Mechanical-Major Equipment and Auxiliary Systems

13.3.8.1 Axial Flow Pumps

13.3.8.1.1 General Design Requirements

The pump equipment should be designed for standby service which is defined as a normally idle piece of equipment that is capable of immediate automatic or manual start-up and continuous operation. The pump equipment including auxiliaries shall be designed and constructed for a minimum service life of 25 years excluding normal wear parts. The estimated average annual operating time should average approximately 1,500 hours annually with the majority of this operating time requiring continuous operation for several days. The fluid description expected during operation includes turbid storm water that may contain sand, silt, and vegetative trash capable of passing the trash rack. Water temperature range should in the range of 80 to 90 degrees F.

The pump should be designed to permit rapid and economical maintenance. ANSI/HI 2.4-2000 provides guidance for the installation, operation, and maintenance of vertical pumps. Major parts, such as the bowl components, should be designed and manufactured to ensure accurate alignment on reassembly. For vertical pumps with bell intakes, the propeller should be removable from bottom of pump bowl without dismantling pump, except for suction bell removal.

13.3.8.1.2 Dynamic Analysis

The pump manufacturer is required to provide the following analysis to ensure the critical speed of the pump does not coincide with the rated operating speed.

13.3.8.1.3 Lateral Critical Speed

The manufacturer shall determine the lateral (dry) critical speed of the pump rotor using static deflection calculations as described in ANSI/HI 9.6.4.2.1 - 2000. A critical speed shall not occur within 25 percent above or below the rated operating speed of the pump.

13.3.8.1.4 Torsional Critical Speed

The manufacturer shall determine the torsional (dry) critical speed of the pump rotor using manual calculation methods as described in ANSI/HI 9.6.4.2.3 - 2000. A critical speed shall not occur within 25 percent above or below the rated operating speed of the pump.

13.3.8.1.5 Lateral Dynamic Analysis

A lateral dynamic analysis shall be performed for an engine horsepower at a rated condition of 335 Hp or greater. Prior to manufacture of any equipment, the pump manufacturer and the engine manufacturer in accordance with the ANSI/HI 9.6.4.2.2 - 2000 shall determine the critical speeds of the equipment in the lateral directions. A natural frequency that occurs within 25 percent above or below the rated operating speed of the pump will not be accepted. The dynamic analysis model shall be constructed using a commercially available program that uses finite element analysis methods. The system shall be analyzed at the run (wet) condition considering the effect of water mass in the column and the damping effect of the highest and lowest sump water levels. The model shall incorporate the critical frequency and mass elastic diagram information provided by the gear manufacturer. The completed dynamic analysis report needs to be submitted to the Engineer prior to start of fabrication.

13.3.8.1.6 Torsional Dynamic Analysis

A torsional analysis shall be performed for an engine horsepower at a rated condition of 335 Hp or greater. Prior to manufacture of any equipment in accordance with ANSI/HI 9.6.4.2.4-2000, the pump manufacturer shall determine the torsional critical speed characteristics of the equipment, including the pump and driver rotational inertias, pump and driver shaft rigidities and inertias and the rigidities of all other rotating equipment in the drive train between the pump and the driver. The analysis shall be performed using a finite element analysis method commercially available with the mass elastic information provided by the pump and gear drive manufacturers. A torsional critical speed that occurs within 25 percent above or below the rated operating speed of the pump and the driver will not be accepted. The completed dynamic analysis report needs to be submitted to the Engineer prior to start of fabrication.

13.3.8.1.7 Pump Components

The following list of pump components is a generalized list and should not be considered a complete and comprehensive description of all the component pieces of a finished pump. It should also be recognized, pump designs vary from manufacturer to manufacturer, and the component descriptions may not be representative of a particular design. The manufacturer shall submit cross sectional drawings indicating the pump design to be supplied with a description of the component pieces of the unit.

13.3.8.1.8 Component Design Criteria

Combined stresses in cast, forged, rolled, or fabricated pressure retaining components, frames and supports shall not exceed that allowed for the given material in Section VIII, Division 1 of the ASME Code. Design pressures for pressure-retaining parts shall be not less than twice the pump's shutoff head at the manufacturer's listed maximum operating speed.

13.3.8.1.9 Base Plate or Support Rings

Most, not all, vertical pumps have a horizontal plate that provides a rigid support for the live and dead loads of the pump and its speed reducer. This steel plate and the supports should be designed in accordance with the AISC Manual of Steel Construction. The pump shall be provided with lifting lugs or eye bolts for handling the pump during loading, unloading, erection, and installation. The support plate or ring should be dimensioned and fabricated to provide a clear opening large enough for removal of the complete pumping unit. The design engineer during the detailed phase of the project shall provide conceptual support dimensions and anchorage details that can be later modified and detailed by the pump manufacturer. It is extremely important the base plate or support ring be properly mounted on and secured to the operating floor or concrete substructure to provide a rigid and uniformly supported foundation for the pump. Often large vibration measurements are a result of an improperly designed and/or installed pump support.

13.3.8.1.10 Gear Pedestal

A pedestal for the vertical pump installations is typically provided and mounted on the base plate or structural steel frame to house the stuffing box and provide support for the speed reducer. Reinforcing ribs and braces shall be provided as required. The pedestal shall be designed to contain water leakage from the shaft packing. A drain shall be provided for return of the leakage water to the sump. Access holes that are sufficiently large and sufficient in number, shall be provided for dismantling the drive shaft coupling, and for maintenance and inspection of the pump seal. The access holes shall be provided with steel guard screens designed for easy installation and removal. The mounting surfaces shall be designed and machined to ensure the reducer can be leveled to satisfy the shaft alignment requirements of the equipment. The pump manufacturer shall closely coordinate with the speed reducer manufacturer to ensure the proper compatibility of the component designs including required installation and alignment procedures. An accurately machined rabbet fit and drilled and tapped holes shall be provided to ensure proper alignment of the reducer to the pedestal.

13.3.8.1.11 Discharge Column and Elbow

The column and discharge elbow shall be designed to withstand the internal pressures and the external loads associated with the pump operation, transportation, erection, or testing required in the field. The elbow and the column section shall be provided with lifting lugs or lifting eyes to facilitate the handling of these parts during installation or maintenance. For small vertical pumps the column and elbow are suspended from the base plate or support frame assembly. For the large installation of this application the elbow and column are supported by the pump from below or a combination of the two. The support of the base plate/support frame and column shall maintain the proper alignment of the pump unit and propeller blade setting. Flanges shall be machined after welding to the pipe to ensure that they are concentric and perpendicular with the axis of the shaft. The column shall be flanged to the diffuser bowl, and shall have rabbet fits or dowels to maintain a concentric alignment. An air/vacuum valve shall be provided in the column pipe to allow the air to escape when the pump is started. The manufacturer shall provide all necessary fittings and cocks for venting and drainage as required. The discharge elbow shall be a mitered type and welded. Turning vanes, if used, shall have clear spacing twice the trash rack spacing. The elbow shall terminate in a plain end with a circular section to allow connection to a standard diameter flexible coupling or, in the case of the large pump alternatives, flanged directly to the wall thimble. Adjustable thrust rods and thrust lugs shall be provided for axial restraint and transfer the load by bridging the coupling to the discharge piping or wall thimble in accordance with ANSI/HI 2.4.4.1 – 2,000-Pipe Supports/Anchors/Joints.

13.3.8.1.12 Diffuser Bowl

The bowl shall be designed to convert the tangential component as well as a portion of the axial component of the velocity into pressure. The diffuser vane design shall optimize the pump performance by straightening and reducing of velocity of the flow as it leaves the propeller. The diffuser vanes should not be an even multiple of the number of propeller vanes to avoid amplification of the pulse energy at the vane passing frequencies. The diffuser bowl shall be flanged and bolted to the discharge column and impeller bowl to prevent rotation and insure proper alignment. Alignment shall be maintained concentric to the pump centerline by use of a rabbet fit or dowels. The bowl shall contain support for the upper propeller shaft bearing.

13.3.8.1.13 Impeller Bowl

Bowl shall be flanged and bolted to the suction bell or FSI support ring and diffuser bowl to prevent rotation. The mating flanges shall have a rabbet fit or dowels to maintain a concentric alignment. The machine finish of the propeller swept area interior of bowl shall be at least 3.2 μm (125 micro-inch) RMS and concentric with propeller axis. Tolerance for concentricity of propeller axis shall not be greater than 20 percent of the operating clearance between the propeller and the bowl.

13.3.8.1.14 Suction Bell

For Alternative 4A a flared suction bell of a design and size to provide constant acceleration to the propeller to prevent flow separation under all operating conditions shall be provided. ANSI/HI 9.8.7.3 - 1998 shall be used as a guide to determine minimum acceptable entrance bell velocity given the minimum required submergence and intake geometry. The bell shall be of a design that has been successfully proven in previous installations compatible with the propeller pump model proposed for the project. The bell shall be made in one piece and flanged to the impeller bowl. The bell shall be supported entirely by the pump casing, supports from the pump floor will not be accepted. Alignment shall be maintained concentric to the pump centerline by use of rabbet fits or dowels.

13.3.8.1.15 Shafts

The shaft shall be designed and manufactured to transmit the maximum torque from the driver to the propeller and support the maximum thrust load with the proper factor of safety. In accordance with ASME B106.1M, the shafts shall be designed for two cases: safety factor of 5.0 based on the ultimate tensile strength of the shaft material and the rated horsepower of the engine; 75 percent of the yield strength of the shaft material and the maximum horsepower of the engine.

All shafts shall be designed to carry the steady state and transient loads suitable for the unlimited number of load applications, in accordance with ASME B106.1M - Design of Transmission Shafting. Where shafts are subjected to fatigue stresses, such as frequent start and stop cycles, the mean stress shall be determined by applying the ASME method for determining the shear endurance limit. The mean stress shall be considered by using the modified Goodman Diagram. The maximum torsional stress shall not exceed the shear endurance limit of the shaft after application of the safety factor of 2 in the endurance limit. The stress concentration factors to account for the geometric discontinuities in the shaft section shall be considered. Fillets in the shaft and the key-ways shall be in accordance with ASME Standard B17.1 - Keys and Key-seats. Petersen's Stress Concentration factors shall be used to determine appropriate stress concentration factors for the shaft system.

The shaft stiffness shall limit deflections under the most severe dynamic conditions over the allowable operating range of the pump in accordance with the performance requirements of the shaft seals and bearings. The running clearances shall be sufficient to ensure dependability of operation and freedom of seizure under all specified operating conditions. All shafts shall be designed to operate within the allowable vibration tolerances in the preferred operating region and ensure the lateral and torsional first critical speeds occurs 25 percent above or below the rated pump speed.

13.3.8.1.16 Sleeves

Unless otherwise specified, renewable sleeves shall be furnished at seal and journal bearing locations. The sleeves shall be held in place by a press fit with locking pins, threaded dowels or other approved methods. The finish of the sleeve shall be at least 16 micro-inch RMS for the seal locations and 32 micro-inch RMS for the guide bearing locations unless otherwise specified by the seal and bearing manufacturers. The pump manufacturer shall coordinate the required machining tolerances with these manufacturers.

13.3.8.1.17 Head Shaft

For the vertical pump alternatives, the manufacturer shall reference the ANSI/HI 2.1.4.2-2,000 standard for the requirements of the hollow/solid shaft driver. The head shaft shall pass concentrically through the speed reducer's hollow shaft allowing vertical adjustment of the propeller. The shaft shall be threaded and shall be provided with a nut to facilitate the adjustment. A circumferential line shall be inscribed or etched on the shaft above the stuffing box and an adjustable pointer shall be provided and mounted opposite this line to indicate a change in vertical position of the shaft and to permit realignment after removal of the speed reducer. The pump manufacturer shall closely coordinate with the speed reducer manufacturer to ensure the proper compatibility of the component designs including required installation and alignment procedures.

13.3.8.1.18 Line Shaft/ Pump Shaft

The shaft design and machining requirements shall be as specified above.

13.3.8.1.19 Shaft Enclosure

A shaft enclosure shall be provided to cover the line shaft and couplings. The enclosure shall be easily assembled and disassembled in the field. External supports or bracing located in the pump water passage shall not be used for support of the tube unless necessary to support intermediate bearings. The enclosure tube shall be designed in accordance with the requirements of bearing water lubrication system to ensure a clean and adequate supply of water to the shaft bearings.

13.3.8.1.20 Shaft Seals

The seal system shall consist of lip-type seal with a stainless steel lip element. All proposed seal systems shall be submitted for review and approval.

13.3.8.1.21 Shaft Couplings

Shaft couplings shall be of a rigid, keyed design of the same material as the shaft and shall allow for the shaft to be readily disassembled, aligned and made up in the field. The coupling shall be capable of transmitting the full load torque required to operate the pump. The finished shaft assembly shall be concentric about the shaft centerline to within 100 μm (0.004 inch). The shaft couplings shall be factory balanced. Couplings furnished shall be shop mounted. The allowable coupling misalignment criterion shall not be used for determination of the parallel offset for the shafts. The axial spacing criteria specified by the coupling manufacturer, however, shall be addressed during installation. All proposed shaft couplings shall be submitted for review and approval.

13.3.8.1.22 Bearings

Hydrodynamic Water Lubricated Journal Bearings: This bearing type shall be used for all line shaft locations including the bowl assembly bearings located immediately above and below the propeller. The line shaft journal bearings shall be nonmetallic synthetic polymer alloy Thordon SXL bearings, manufactured by Thordon Bearings Inc., Ontario, or an approved equal. The bearing design shall be the responsibility of the bearing manufacturer to ensure the proper diametral clearance necessary for the satisfactory lubrication and the appropriate housing to support the bearing. The surface roughness and hardness of the shaft (sleeve) shall be in accordance with the bearing manufacturer’s recommendations. The bearing shall be easily removable for servicing in the field. The bearings shall be of sufficient number and length to ensure permanent shaft alignment and to prevent shaft whip and vibration. The manufacturer shall ensure the alignment of the shaft bearings is in accordance with the requirements of the bearing manufacturer. The alignment shall include the fit of the bearing in the housing as well as on the shaft, (sleeve). The alignment requirements and procedures shall be submitted as part of the operation and maintenance manual.

13.3.8.1.23 Bowl Assembly Bearings

The bowl assembly bearings are the bearings on the discharge and suction side of the propeller. The suction side bearing shall be protected by a sand cap, to prevent sand or grit from entering.

13.3.8.1.24 Thrust Bearing

Thrust axial loads, (up-thrust and down-thrust), shall be supported by a thrust bearing(s) in the speed reducer. The thrust bearing type shall be an angular contact ball or spherical roller. The bearing shall be sized for continuous operation under all specified conditions and shall provide full load capabilities if the pump’s normal direction is reversed. The pump manufacturer shall closely coordinate with the speed reducer manufacturer to ensure the proper compatibility of the component designs.

13.3.8.1.25 Stuffing Box

Provide stuffing box to prevent leakage through casing. The packing gland shall be easily serviced by access through openings in the pedestal. A grease lubricated packing gland split longitudinally to facilitate removal and renewal without entering pump from below operating platform shall be provided. A shaft sleeve shall be provided at seal location.

13.3.8.1.26 Propeller

Provide axial flow propeller of smooth construction for maximum efficiency. Generally, for specific speeds between 11,000 and 15,000, three vanes have been found to be the most efficient. Propeller shall be removable from bottom of pump bowl without dismantling pump, except for suction bell removal. Securely lock propeller with longitudinal and annular keys to prevent rotational and axial movement. Collets or taper fits will not be acceptable. The propeller shall be factory balanced. Because of the relatively low head produced by axial flow pumps, the skin friction or drag is of greater importance than for centrifugal or mixed flow pumps. Therefore, a high degree of impeller vane streamlining and polishing is required to obtain the optimum peak efficiency.

The impeller hub ratio, the hub diameter to the impeller outside diameter, is an important design parameter for axial flow pumps. This ratio is established experimentally. In higher specific speed pumps the small the hub diameter, the greater the free area for flow and therefore the greater capacity and lower head.

13.3.8.1.27 Materials of Construction

The material selections for the pump components are determined by the environmental conditions of the application. ANSI/HI 9.3.2-2,000 provides guidelines of for corrosion, erosion, and abrasion resistance for fresh water applications. The materials specified shall be considered the minimum acceptable in accordance with these guidelines or that required by the SFWMD. The pump manufacturer shall have the opportunity to propose alternative materials for the purpose of providing greater strength or to meet required stress limitations. However, these proposed alternative materials must provide at least the same qualities as those specified for the purpose. All material should conform to the latest ASTM specification or other listed commercial specifications covering the class or kind of material to be used. Table 13.3-13 provides the proposed material specification for various components of the pump for this fresh water application.

Table 13.3-12 Pump Component Material Specifications

Item	Material
Base plate/support frame	Carbon steel – ASTM A36
Discharge column and elbow	Carbon steel – ASTM A283 grade C or A516 grade 70
Gear pedestal	Carbon steel – ASTM A36
Impeller and diffuser bowls	Cast iron – ASTM A48 class 30
Suction bell	Cast iron – ASTM A48 class 30
Shafts	Stainless steel – ASTM A276 type 316L
Shaft couplings	Stainless steel – ASTM A276 type 316L
Shaft enclosure	Stainless steel – ASTM A276 type 316L

Item	Material
Shaft sleeves	Stainless steel – ASTM A276 type 316L
Propeller	Copper alloy castings – ASTM B584-C87500
Packing gland	Stainless steel – ASTM A743 type 316L
Nuts, bolts, dowels, keys, fasteners	Stainless steel – ASTM A-193 type 316L

13.3.8.1.28 Metalwork

The quality of the manufacturing of axial flow propeller pumps is very important to ensure the specified hydraulic performance is attained as well as ensure a well made pump that will be reliable and provide a long service life. Visits should be made by the SFWMD's representative to the manufacturer's plant at key points in the fabrication of the pump to inspect the workmanship including metalwork and materials. The SFWMD's representative should check mill certifications to ensure proper materials have been used, check the propeller templates to ensure they comply with those modeled, and generally inspect all aspects of the fabrication to ensure it complies with the contract specifications as well as standard industry practice.

13.3.8.1.29 Welding

Welding of piping, pressure containing parts and wetted parts, as well as repair to such parts shall be performed and inspected by operators and procedures qualified in accordance with Section VIII, Division 1, and Section IX of the ASME Code. All repairs and repair welds shall be properly heat treated and nondestructively examined for soundness and compliance with the applicable qualified procedures. Repair welds should be nondestructively tested by the same method used to originally qualify the part. Unless otherwise specified, all welding other than that covered by Section VIII, Division 1 of the ASME Code and ANSI/ASME B31.3, such as welding on base plates, non-pressure ducting, and lagging should be performed in accordance with ANSI/AWS D1.1. Pressure containing welds, including welds of the casing to horizontal and vertical joint flanges should be full fusion, full penetration welds. Auxiliary piping welded to alloy steel casings should be of a material with the same nominal properties as the casing material or shall be of a low carbon austenitic stainless steel.

13.3.8.1.30 Flame Cutting

Flame cutting of material other than steel should be subject to approval. Shearing should be accurately done and all portions of work neatly finished. Steel may be cut by mechanically guided or hand-guided torches, provided an accurate profile with a smooth surface, free from cracks and notches is secured. Surfaces and edges to be welded shall be prepared in accordance with ANSI/AWS D1.1. Chipping and/or grinding should not be required unless necessary to remove slag and sharp edges of mechanically guided or hand guided cuts not exposed to view. Visible or exposed hand guided cuts should be chipped, ground, or machined to metal free of voids, discontinuities, and foreign materials.

13.3.8.1.31 Minimum Thicknesses

Minimum wall thickness should conform values presented in Table 13.3-14.

Table 13.3-13 Pump Components Minimum Wall Thickness

Wall Thickness (Inches)			
Pump Size	**Impeller and Diffuser Bowls**	**Suction Bell**	**Discharge Column and Elbow**
24	1/2	3/8	1/2
30	1/2	3/8	1/2
36	5/8	1/2	5/8
42	5/8	1/2	5/8
48	5/8	1/2	5/8
54	3/4	5/8	3/4
60	3/4	5/8	3/4
72	3/4	5/8	3/4
84 and greater	3/4	5/8	3/4

13.3.8.1.32 Alignment of Wetted Surfaces

The manufacturer should exercise care to ensure that correct alignment of the wetted surfaces being joined by a flanged joint is being obtained. Where plates of the water passage change thickness, transition should occur on the outer surface, leaving the inner surface properly aligned. When welding has been completed and welds have been cleaned, but prior to stress relieving, joining of plates should be carefully checked for misalignment. Localized misalignment between inside or wetted surfaces of adjoining flange connected section of the pump should not exceed the amount shown in Table 14 for the respective radius or normal distance from the theoretical flow centerline. Misalignment greater than the allowable amount needs to be corrected by grinding away the offending metal, providing the maximum depth to which metal is to be removed does not exceed the amount specified in Table 13.3-15.

Table 13.3-14 Pump Components Misalignment Tolerances

Diameter	Wall Thickness	Max. Allow. Offset	Max. Removal Depth
24	3/8	1/16	3/32
30	3/8	1/16	3/32
36	3/8	3/32	3/32
42	1/2	3/32	1/8
48	1/2	1/8	1/8
54	1/2	1/8	1/8
60	3/4	5/32	3/16
72	1	5/32	3/16
84 and greater	1-1/8	3/16	1/4

The manufacturer should finish all corrective work by grinding the surfaces to a smooth taper. The length of the taper along each flow element needs to be ten times the depth of the offset error at the flow line. Wetted surface irregularities that might have existed in an approved model should not be reason for accepting comparable surface irregularities in the pump.

13.3.8.1.33 Castings

Castings must be sound and generally free from porosity, hot tears, shrink holes, blow holes cracks, scale, blisters and similar defects. The engineer or better yet his expert representative should carefully examine all castings for surface defects. Defects found should be further examined by a nondestructive means. The manufacturer's examination personnel shall be certified in accordance with applicable ASTM requirements. The examination procedure should be one of the following suited for the application:

- ASTM A 609/A 606M - Castings, Carbon and Low Alloy, and Martensitic Stainless Steel, Ultrasonic Examination Thereof, 1991,R 1997
- ASTM E 709 - Magnetic Particle Examination, 1995
- ASTM E 165 - Liquid Penetrant Examination, 1995

Minor surface imperfections should be filled or ground down as necessary to preserve the correct contour and outline of the propeller and to restore the surface to the same degree of finish as the surrounding surfaces. Mold parting fins and remains of gates and risers are be chipped, filed or ground. The manufacturer should correct surface pits, depressions, projections, overlaps showing greater the 1/16-inch variation from the general contour for that section. Castings that exhibit surface imperfections covering an area more than 10 percent of the blade surface should be rejected. In addition, casting defects should not exceed 12.5 percent of the total blade thickness. Where defects do not affect the strength and serviceability of the casting, casting defects may be repaired by an approved welding procedure. Steel castings may be repaired by using a qualified welding procedure based on the requirements of Section VIII, Division 1 and Section IX of the ASME code. Iron castings may be repaired by plugging within the limits of the applicable ISO (ASTM) specification. The surfaces of accepted castings are then cleaned by sandblasting, shot blasting, chemical cleaning, or any other standard method to meet the visual requirements of MSS-SP-55.

13.3.8.1.34 Flanges

Flanges should be provided on the major components of the pump casing, (suction bell, impeller bowl, diffuser bowl, discharge column). The flange should be dimensioned in accordance with AWWA C207, Class B and drilled ANSI/ASME B16.1, Class 25. The design of the flanged joints shall be air and water tight without the use of preformed gaskets. The flanges shall be parallel machined and mounted parallel to a plane that is normal to the pump shaft centerline. Flanges on each end of the same component shall have a parallel tolerance of 0.002 inch. The finish machine mating surface on the flange shall be to 3.2 μm (125 micro-inch) RMS finish or better. The mating flanges should be provided with a male-female rabbet or with not less than four tapered dowels equally spaced. If a rabbet fit is not provided, the manufacturer shall provide the method used to determine concentricity of the connected pieces. The flanges shall be connected by a least two continuous fillet welds. One weld shall connect the inside diameter, the other the outside diameter. External stiffeners should be provided if required.

13.3.8.1.35 Balancing

Unbalance is the most common source of vibration. Vibration due to unbalance occurs at a frequency of 1 x rpm of the unbalanced element, and its amplitude is proportional to the amount of unbalance. Balance can be either static or dynamic. Static unbalance is defined as an unbalance where the center of rotation is displaced parallel to the geometric center of the rotating element. Single plane balancing is only successful in conditions where the unbalance is pure static. Dynamic unbalance the most common type of unbalance is defined as unbalance where the axis of rotation does not coincide or touch the element's geometric centerline. This type of unbalance requires correction in two or more planes. The correction must always result in the sum of the forces from the correction weights and the unbalance equal zero as well as the sum of the moments about the unbalance must equal zero. Sources of unbalance can be numerous. Couplings can have missing washers, excess key material, uneven length bolts, or the bolt in backwards. A buildup of dirt or debris can contribute to vibration. In accordance with ANSI/HI 9.6.4.5.1-2000 "Unbalance of Rotating Parts and Rotor Balance": Component balance shall be a single plane spin balance to ISO 1940-1986 E balance quality grade G6.3. When the ratio of the component outside diameter divided by the distance between correction planes is less than six, a two plane balance shall be required. The propeller component shall be balanced at its rated speed.

13.3.8.1.36 Quality of Balance Guidelines

The balancing equipment and procedures shall follow the guidelines of ANSI/HI 9.6.4.5.1 - 2000. The manufacturer should be required to submit the proposed balancing procedures and balancing equipment data.

13.3.8.1.37 Allowable Unbalance

The allowable unbalance for the rotating parts of the pump and its propeller shall be determined in accordance with the procedure defined in ANSI/HI 9.6.4.5.1 - 2000.

13.3.8.1.38 Balance Correction

The material removal or addition for balance correction shall follow the guidelines in ANSI/HI 9.6.4.5.1 - 2000. The Manufacturer should submit the procedure proposed to correct the unbalance of the component.

13.3.8.1.39 Assembly

Pumps shall be completely shop assembled, (vertical pumps in the vertical position), and aligned prior to shipping. Tolerances shall not exceed those specified or shown in the manufacturer’s drawings. Rotating elements shall be checked for binding. The SFWMD’s representative should witness the shop assembly. After completion of the specified factory tests, pumps shall be prepared for shipment with the minimum amount of disassembly and such that no field disassembly, cleaning or flushing is required. Any components removed for shipping shall be match-marked prior to removal and shipment.

13.3.8.1.40 Pump Bearing Lubrication System

The pump shaft bearings shall be lubricated with water from the fresh water supply system comprised of two centrifugal pumps, one of which is redundant, and interconnecting piping, valves, instruments and controls. Two lubrication water pumps will draw water from the fresh water storage tank and pump the water to the water lubricated bearings of all the pumps in the station. The fresh water will be transported to the individual units through a manifold system. Each pump will have a solenoid valve, which when activated, will allow water to flow to the pump. In addition, a water constituent sample of the water supply source should be taken and results given to vendors for their review of compatibility with their equipment. All piping shall comply with ASTM A 53 or ASTM A 106 steel pipe. Pipe smaller than 2 inches in diameter shall be schedule 80. Pipe 2 inches in diameter or larger shall be schedule 40. Flanges shall comply with ASTM A 234 Grade WPB or WPC, Class 150 or ASME B16.11, 3000 pounds. Threaded fitting shall be in accordance with ASME B16.3, Class 150. Valves shall be in accordance with MSS SP-80, Class 150.

13.3.8.1.41 Factory Performance Test

The pump manufacturer shall conduct a factory performance test to demonstrate the pump complies with the specified performance. The test shall be in accordance to ANSI/HI 2.6.5 - 2000 "Vertical Pump Test." The test should be conducted by a registered professional engineer employed by the manufacturer and witnessed by the owner's representative. The tests shall be in accordance with the following standards:

- The test instrumentation and test setup shall be in accordance with ANSI/HI 2.6.5.4 - 2000 and ANSI/HI 2.6.5.5 - 2000
- Measurement of the rate of flow shall be in accordance with ANSI/HI 2.6.8 - 2000. The flow measuring system shall have an accuracy of 1.5 percent at BEP
- Measurement of pump discharge and suction heads shall be in accordance with ANSI/HI 2.6.9 - 2000
- Measurement of pump input power shall be in accordance with ANSI/HI 2.6.10 - 2000
- Measurement of the rotary speed shall be in accordance with ANSI/HI 2.6.11 - 2000

The test shall also comply with the following specifications:

- Test throughout preferred operating region at the rated speed
- The pump test data shall be taken under steady state conditions
- The maximum permissible short-term speed fluctuation shall be no more than 0.3percent

Calculations shall be in accordance with ANSI/HI 2.6.5.8 - 2000. Results shall be plotted in accordance with ANSI/HI 2.6.5.9 - 2000 and shall develop the following performance curves; rate of flow (gpm) vs. pump total head (feet); rate of flow (gpm) vs. pump input power (Hp) and rate of flow (gpm) vs. bowl assembly efficiency (percent). The manufacturer shall provide the calculation of the internal pump losses. The curves shall stamped as certified correct by a registered professional engineer.

13.3.8.1.42 Factory Inspections

The owner's representative should make periodic visits to the manufacturer's plant to inspect the fabrication and assembly of the pump. The manufacturer shall have available for review detailed fabrication and assembly drawings for manufacture of the entire pump. Drawing details shall include all dimensions, tolerances, shaft clearances, bearings, diameter and tip clearance of the propeller, couplings, and packing gland. The manufacturer shall also have available at the time of the inspections purchase orders, mill orders, or shop orders including certified material test reports that indicate components and/or materials to be used in the pump's manufacture. The owner's representative shall review these records, drawing details and the pump unit for compliance with the specifications as well as standard industry practice.

13.3.8.1.43 Field Operating Test

An operating test shall be conducted to determine that the performance of equipment and controls. The tests shall consist of placing all pump equipment in operation under the pumps preferred operating region for a period of four hours. If the water conditions are not suitable at the time of the test, the test shall be postponed until conditions are acceptable.

13.3.9 Diesel Engine Drivers

13.3.9.1 General Description and Design Requirements:

Flood control pump stations require a reliable driver that will function through severe storm conditions as well as many days after the event. Typically diesel engines are the most reliable and efficient drivers for this application. Electric service should not be considered a reliable power source. There may be justification to utilize electric motor drivers with diesel engine generators when the horsepower requirements are relatively small and the station is utilized extensively for drainage or water supply throughout the year. A life cycle cost analysis should be developed for selection of the most cost effective driver alternative.

For vertical pumps the diesel engine drives the shaft of the axial flow single stage propeller pump through a right angle speed reducing gear transmission. The engine is connected to the reducer's horizontal input shaft by two universal joints and an intermediate shaft. A power take-off and clutch assembly is typically provided to disengage the engine for service. The engine can be remotely monitored and operated via telemetry or other communications facilities. Local manual starting and monitoring is provided. The engine's electronic control module typically output engine parameters and functions via a data-link to a logic controller.

The engine selected typically is a compression-ignition type, four cycle, diesel engine for stationary applications. The engine can be either a vertical in-line or vee piston type, and have a solid cast block with not less than six cylinders. The engine can be naturally aspirated or turbocharged-after-cooled. Engines should be current models of a type in regular production with all devices specified or normally furnished with the engine. The available models of drivers shall be matched to the pumping system requirements. The engine model proposed shall be a unit with a satisfactory service record of not less than 36 months of operating 1,200 hrs/yr under similar or more severe conditions of duty. The engine should be provided and installed complete and totally functional, with all necessary ancillary equipment including, but not limited to: air filtration; starting system; fuel system; cooling system; instrumentation; and engine exhaust system. The engine can be electric starting or for larger models compressed air and can be either cooled via a heat exchanger or a flow through system or for smaller Hp models by the factory provided radiator.

Engines of equal horsepower do not necessarily result in equal performance. Horsepower, which is an arbitrary concept derived by James Watt, is a measure of an engine's ability to move 33,000 pounds one foot in one minute. The other component of an engine's output is torque, with the units of foot-pounds. In concept, torque is simply the twisting force on a bolt resulting from a one foot long wrench with a one pound weight at its end. Torque is the force that turns the engine's crankshaft and is the critical criteria of the power train analysis. Torque is directly proportional to engine displacement. Torque can be directly measured by a dynamometer, horsepower can not be directly measured. The formula for conversion:

$$\text{Horsepower} = \text{Torque} \times \text{rpm} / 5252$$

Therefore it is important to compare horsepower/torque curves for the proposed engines. The objective is to achieve maximum torque for the operating range of the engine. The flatter the torque curve through this range of operation the better since the engine will be operating at or near maximum torque. If the operating speed is set, for example at 1,800 rpm, then the engine torque at this rpm should be at or near maximum.

The output power to be delivered by the engine should be based on the input power required by the pump and transmission though out the pump curve from shut off head to the maximum operating flow range as determined by the pump manufacturer. The engine shall not be over loaded through pump’s allowable operating region. The engine’s output power shall be determined by the engine manufacturer in coordination with the pump manufacturer. It should be noted the engine power ratings are based on the total power output capability at the flywheel. The required engine output shall include the horsepower requirements of the engine auxiliaries.

The engine will generally be started with the pump engaged. The engine manufacturer in coordination with the pump manufacturer, shall ensure the engine proposed has adequate accelerating torque under full load start-up conditions, (additional torque required above normal operating torque), for the pump to attain the rated speed in a reasonable amount of time. The engine manufacturer shall also ensure there are no damaging overload conditions during the engine’s warm-up period.

13.3.9.1.1 Engine Rating

The engine service shall be “Continuous Duty” intended for continuous use for load application requiring uninterrupted service at full power. The standard reference conditions, methods of declaring the power, fuel consumption, lubricating oil consumption, and test methods for diesel engines is in accordance with applicable sections of ISO 3046 for the conditions listed below. The basis for gross engine power rating, methods for correcting observed power to reference conditions and the method for determining gross full load engine power with a dynamometer is SAE J1995.

13.3.9.1.2 Project Site Conditions

- Maximum air temperature: 105 degrees F
- Minimum air temperature: 35 degrees F
- Maximum raw water temperature: 90 degrees F
- Minimum raw water temperature: 60 degrees F

- Elevation: sea level
- Relative humidity: 80 percent

13.3.9.1.3 Engine Speed at the Rated Condition

The engine speed should be selected by review of the available engine models for the required horsepower/torque range of operation. For small to medium sized engines, (< 600 Hp) the rated speed is typical 1,800 to 2,100 rpm. The engine speeds for the larger 1,000 cfs pump will probably be in the range of 1,000 rpm.

13.3.9.1.4 Fuel Requirements

Again the designer is limited to the standard engine models which operate on 2 or 2-D, (regular) diesel fuel oil, 40 cetane, (minimum), ASTM D396 and ASTM D975.

13.3.9.1.5 Fuel Consumption

The standard reference conditions and methods of declaring fuel consumption shall be in accordance with applicable sections of ISO 3046. Typically the fuel consumption rate shall not exceed 0.45 pounds per bhp-hour between 75 percent and 100 percent of rated full load for the following conditions:

- Fuel heat value: 19,350 BTU
- Unit elevation: sea level unless otherwise noted.
- Intake air temperature: 90 degrees F
- Barometric pressure: greater than 28.25 inches mercury

13.3.9.1.6 Emissions Requirements

The EPA emissions regulations for stationary diesel engine applications are being drafted for the finished installation to comply with EPA Tier 2, Stage II emissions requirements. Engine manufacturer's are well aware of these requirements and have had to deal with them for their on-road engines. This technology has on many models been adapted to the modern stationary production models. The designer should specify the Environmental Protection Agency (EPA) regulation requirement to ensure the Contractor satisfies the most current EPA requirements. The engines used in this analysis are Tier I compliant.

13.3.9.1.7 Engine Electronics

The engine's electronic control module provides monitoring of vital engine parameters and control of engine operation. The system regulates emissions and optimizes fuel economy and provides condition monitoring to prevent engine damage. The electronic control system has programmable speed control. The electronic package also provides standard data-link to a logic controller for manual and remote monitoring and operation via telemetry facilities. A standard factory supplied engine control and monitoring panel can be specified for manual operation.

13.3.9.1.8 Rotation

The rotation of the engine should be the SAE standard rotation with the speed reducer and pump to match this rotation. This direction, looking towards the front of the engine, is anti-clockwise.

It is intended that the engine deliver power in one direction only and an anti-reverse rotation device shall be provided by the reduction gear to prevent reverse rotation by the backflow of water through the pump at shut down.

13.3.9.1.9 Engine Mounting

For the small and medium sized engines, (< 600 Hp), the engine and all accessories should be skid mounted. Complete equipment foundation plates, sole plates, mounting straps, brackets or structural bases with suitable anchor bolts, nuts, sleeves, washers and shims or wedge plates and vibration dampers or isolation blocks should be furnished as required. Resilient mounts should also be provided and should be capable of fully restraining the engine and limiting its motion under acceleration induced forces and torque reactions. The engine mounts shall be capable of alignment and leveling. These mounts are standard designs provided by the engine manufacturer. It is important the Contractor coordinate the foundation requirements with the various trades for the anchorage and foundation details to be provided.

13.3.9.1.10 Exhaust System

A complete and separate exhaust system should be provided for each engine. The engine exhaust system piping should be provided and laid out with the shortest and straightest runs possible consistent with the location of the exhaust silencers in relation to the engines. Sharp bends shall be avoided by the use of long sweep fittings wherever practical. Horizontal sections of the piping shall be sloped downward away from the engine to a condensate trap and drain valve.

13.3.9.1.11 Piping

All piping should be 304 stainless steel in accordance with ASTM A 240/A 240M. All pipe sections should be flanged where practical. Piping smaller than two inches in diameter should be Schedule 80. Piping with a diameter of two inches or larger should be Schedule 40. The vertical exhaust piping shall be provided with a hinged, gravity-operated, stainless steel, self closing cap. Thermal expansion and/or vibration shall be addressed by a short length or lengths of an approved multi-ply stainless steel bellows type flexible sections at each engine. Suitable stainless steel sleeves with retainer rings should be provided together with suitable packing for wall penetrations to allow free movement of the pipe in accordance with NFPA 37.

13.3.9.1.12 Supports

Pipe supports for the exhaust lines and braces for the, exhaust silencer, and tailpipe need to be provided as necessary. Pipe hangers shall be in accordance with MSS SP-58 and MSS-69. The designer may want to provide details of the supports and hangers in the construction contract drawings to ensure a quality installation.

13.3.9.1.13 Exhaust Silencer

The designer needs to review the local noise ordnances to ensure compliance of the proposed installation. Typically the noise level taken three feet from the silencer shall not exceed 86 dBA. The exhaust silencer should be at a minimum a critical grade chamber type exhaust muffler mounted on the exterior of the pump station building. The exhaust silencer, support, and miscellaneous fasteners should be ASTM A276 type 304 stainless steel. The designer should provide the silencer support details. The engine manufacturer needs to provide input to the

Contractor for the proper selection of the silencer that will provide the most effective system considering; noise levels generated, pressure drop and physical size of the silencer.

13.3.9.1.14 Exhaust Line Insulation

All exhaust lines for the engines inside the building need to be insulated with not less than three inch thickness of ASTM C 533 calcium silicate insulation. The insulation shall be secured with stainless steel bands and covered with an aluminum jacket. The aluminum jacket should overlap not less than three inches longitudinal and circumferential joints and should be secured by bands at not more than 12-inch centers. Longitudinal joints shall be overlapped down. Circumferential joints should be sealed with a coating that is recommended by the insulation manufacturer. Aluminum should be smooth sheet 0.016-inch nominal thickness and have a factory applied polyethylene and kraft paper moisture barrier. At pipe flanges and expansion joints, the insulation at each side of the flanged connection should be tapered for a short section to permit removal of bolts without disturbing the insulation.

13.3.9.1.15 Air Intake System

A complete and separate air intake system shall be provided for each engine. The contractor shall be responsible for the design and installation of the air intake system in accordance with the engine manufacturer's requirements and the project site conditions specified above.

13.3.9.1.16 Air Intake Filter

The air intake filter for each engine shall consist of high-efficiency, washable paper elements packaged in a low restriction waterproof housing. The filter shall be provided in a location convenient for servicing.

13.3.9.1.17 Inline Silencer

For turbo-charged engines, an inline silencer shall be provided on the air intake. The silencer shall be of the high frequency filter type. A combined filter silencer unit meeting the requirements for the separate filter and silencer items may be provided.

13.3.9.2 Engine Cooling System

The cooling system for smaller horsepower engines are typically the manufacturer's standard radiator cooling system with an engine driven fan. The air flow shall be from the engine to the radiator and exhaust out through a shroud to the exterior of the building. Ambient air conditions within the building will vary from 60 degrees F to 105 degrees F. The contractor shall provide a flexible shroud that is securely fastened to the radiator and adjacent perimeter of the wall of the exhaust opening. The shroud shall be provided in accordance with manufacturer's recommendation and shall be easily removed for maintenance.

For larger engines where radiators are not practical a complete and separate cooling water system should be provided. The system can either be a closed or flow through system depending on the engine size and the cost of the systems. The decision of the cooling system type shall be determined at a later design phase. For any system specified, the contractor shall be responsible for the details of the design and installation of the system in accordance with the engine manufacturer's requirements and the project site conditions specified. For this analysis it was assumed the cooling water system is a flow through system with the cooling water provided by the station's service water system which includes turbine pumps and a filtration system

The cooling water system for each engine should operate automatically while the engine is running. The closed cooling water system typically have an engine driven jacket water pump, a submerged pipe heat exchanger (Keel Cooler or equal), expansion tank, and an automatic temperature regulating valve. The cooling system shall be designed for the maximum raw water temperature and the maximum ambient temperature. The system circulates jacket coolant through the engine at the temperature and flow rate recommended by the engine manufacturer. The coolant is typically an ethylene-glycol water mixture. The engine driven jacket water pump forces water through the engine cooling passages, the heat exchanger, expansion tank, and back to the pump. The pump is typically the manufacturer’s standard centrifugal type pump properly sized for the intended purpose.

For the closed system, each engine cooling system shall include pipe or coil submerged type heat exchanger, (Keel Cooler or equal) located on the wall of the intake. The heat exchanger shall be of ample capacity to match the engine with maximum water temperature in the intake. The jacket water shall flow from the engine to the cooling coils and then to the expansion tank before returning to the jacket water pump inlet. The temperature rise of the coolant across the engine shall not exceed the recommendations of the engine manufacturer.

Each engine cooling system shall include one thermostatically controlled proportioning valve of appropriate size and temperature rating installed at the after cooler and bypass line. The valve shall be complete with automatic control element. A bypass with an automatic temperature regulator shall be installed around the heat exchanger so that the temperature of the jacket water may be regulated.

Each engine shall be equipped with a coolant temperature sensor and coolant level sensor. The temperature sensors shall provide signals for coolant temperature indication and high coolant temperature alarms.

A jacket water expansion tank shall be furnished for each engine. The tank shall be of welded steel construction and shall be hot dipped galvanized inside and out after fabrication. The tank shall have a capacity of not less than 10 gallon and shall be suitable for an operating temperature of 250 degrees F and a working pressure of 125 pounds per square inch gauge (psig). The tank shall be tested and stamped in accordance with ASME BPV VIII Div 1 and registered with the National Board of Boiler and Pressure Vessel Inspectors. The tank shall be properly fitted for vent, overflow, expansion, and make-up lines and mounted so the bottom of the tank is above the top of the engine. A brass water gage with valves shall be provided on the tank. The Contractor shall submit the details of the tanks support for approval.

13.3.9.3 Oil Lubrication System

The engine lubricating oil system shall be of the manufacturer's standard design for the model engine proposed. The lubricating system shall be monitored and controlled by the engine’s electronic control system to insure proper lubrication for the application proposed. The system shall be readily accessible for service such as draining and refilling. Each system shall permit addition of oil and have oil-level indication. All items of equipment shall be furnished and installed as complete units ready for operation.

13.3.9.3.1 Lube Oil Sensors

Each engine shall be equipped with lube-oil temperature and pressure sensors. The temperature sensors shall provide signals for high lube-oil temperature indication and alarm. In addition, low lube-oil pressure indication and alarm sensors shall be provided.

13.3.9.3.2 Lubricating Oil Filter

Each engine lubricating oil system shall include a suitable lubricating oil full-flow, duplex (80) micron filter of the throw away cartridge type. The filter medium shall be absorbent type as recommended for use with the type of oil used in the engine. The filter shall be readily accessible and capable of being changed without disconnecting the piping or disturbing other components. The filter shall have the inlet and outlet connections plainly marked.

13.3.9.4 Starting System

For this analysis it was assumed the engines would be started by a compressed air system via an air motor supplied by the engine manufacturer. However, it is suggested to next preliminary design stage investigate the economic advantages of an electric starting system consisting of a 24 VDC battery starting system, manually (or remotely started), from the engine control panel. The engine direct current starting system would separate from the engine control panel. The starting system shall be designed to have sufficient capacity to start the engine with the pump engaged. Starting motors are in accordance with SAE ARP 892.

For an electric starting system, a starting battery system is provided, one system for the station, which includes batteries, battery charger with over-current protection, battery rack, inter-cell connectors, spacers, metering, and relays. The simpler option is to follow the design of the auto industry with a separate starting battery for each engine and an alternator to recharge the battery while the engine is running. The lead acid type battery shall meet or exceed the requirements of SAE J537. A standard requirement of the battery sufficient capacity to provide the minimum cranking cycle consisting of no fewer than three cranking periods of up to eight seconds per period with eight second intervals between crank periods or shall be sized in accordance with the engine manufacturer’s requirements.

The battery charger shall have a current limiting 10 ampere battery charger, conforming to UL 1236, and shall be provided to automatically recharge the battery bank. The charger shall be capable of providing both automatic float charging and equalizing charging of the battery installation. The battery charger shall be capable of providing a floating charge rate for maintaining the batteries in a fully charged condition. An ammeter and voltmeter shall be provided on the charger to indicate charging rate and voltage. The charger shall have alarm functions providing indications of low battery voltage, high battery voltage, and battery charger malfunction.

13.3.9.5 Power Take-off and Clutch Assembly

For this analysis a power take-off (PTO) and clutch assembly was not provided. The final decision to include a PTO shall be made at a later design phase. The clutch, if provided, shall be a mechanical type that will allow the engine shaft to be disengaged manually from the drive shaft. The PTO and clutch assembly shall be rated for the classification duty required by the engine and pumping system and shall be chosen by the engine supplier to best suit the selected engines. Note, the engine will generally be started with the pump engaged. Disengaging will typically occur prior to servicing of the engine or the pump. Lubrication of the clutch bearings, if required, shall be done without removal of the unit.

13.3.9.6 Drive Shaft and Coupling Assembly

For small horsepower applications the speed reducer is typically connected to the driver by a carden shaft and double, heavy duty, needle bearing type universal joints. The bearings should have minimum rating of a B10 life of not less than 16,000 hours (including applicable service factor for driver utilized) and shall have a service factor of two based on the maximum rated load. In addition, at maximum overload conditions, the stresses shall not exceed 80 percent of yield strength. Universal joints shall have forged steel yokes and spiders and shall have sealed needle roller bearings. Universal joints shall be installed in pairs. The angle between each shaft and the intermediate shaft shall be equal and not exceed the manufacturer's recommendations. The driving pins on the yokes attached to the intermediate shaft shall be set parallel to each other. The universal joints shall be dynamically balanced to AGMA balance classification seven or better and shall be grease lubricated unless self-lubricated.

To address torsional vibration, rubber torsional coupling between the engine/PTO output shaft and the carden shaft is recommended.

13.3.10 Speed Reducers

For vertical pumps to transmit the power from the horizontal shaft of the engine to the vertical shaft of the pump a right angle gear is required. To reduce the output shaft rotary speed of the engine to the input shaft speed of the pump requires a speed reducer. These functions are combined in one unit a right angle speed reducer. Right angle speed reducers perform the following functions:

- Transmit the power from the diesel engine driver to the vertical axial flow pump
- Redirect the power from the horizontal shaft of the driver to rotate the vertical shaft of the pump
- Reduce the speed of the shaft rotation of the engine driver to the required rpm of the pump shaft
- Prevent rotation of the pump shaft from backflow of water after shutdown of the driver
- Provide a thrust bearing(s) to address the up-thrust and down-thrust hydraulic loads of the pump

Speed reducers are standard products of manufacturers and conform to conform to AGMA standards. The furnished unit should display the AGMA insignia as evidence of conformance to the requirements of AGMA 6010-F97 or AGMA 6025-D98. Standard practices shall be as defined and set forth by the American Gear Manufacturer's Association. The procedure outlined in AGMA 2005-C96 and AGMA 6010-F97 shall be followed.

The more detailed description of the typical speed reducer used in the vertical pump application is a single reduction right angle spiral-bevel gear. The reducer's low speed output shaft is of a hollow shaft design. This arrangement permits the pump head shaft to pass concentrically through the reducer shaft for vertical adjustment of the pump propeller. The reducer's high speed input shaft shall is connected to the driver by two universal joints and an intermediate shaft.

Bevel gears are used to connect shafts whose axes intersect. Spiral bevel gears have obliquely curved teeth with a spiral angle such that the face advance is greater than the circular pitch. This results in a continuous pitch line contact in the plane of axes of the gears. The contact between teeth begins at one end of the tooth and progresses obliquely across the face of the tooth. This results in a smooth action suitable for high speed operation.

For horizontal pumps a parallel shaft gear drive is required.

13.3.10.1 Performance Requirements

The performance requirements of the reducer are determined by coordination with the pump and driver requirements. The unit's primary function is to transmit the necessary torque from the approved driver to the pump shaft for the entire operating range of the system. A service factor of 2.0 shall be applied to the manufacturer's published rating. The reducer's shaft output speed is designed to equal the pump rotary speed at the rated condition. The overall reduction ratio shall properly match the driver speed with the pump rpm at the pump's rated condition. The rotation of the input shaft of the speed reducer should match the typical rotation of the driver.

The reducer should have a continuous mechanical horsepower rating of not less than 150 percent of the horsepower rating of the engine driver. The pump input power (Pp) at the rated condition is defined by the requirements of the vertical pump. The reducer shall be designed with sufficient capacity to stall the driver without injury to the reducer. The reducer includes a thrust bearing(s) to address the up-thrust and down-thrust hydraulic loads of the pump. Speed reducers have efficiencies in the range of 96 percent. Before assembly, each gear and shaft assembly shall be dynamically balanced in accordance with ANSI/AGMA 2005-C96.

13.3.10.2 Operating Conditions

The pump manufacturer should obtain or develop the following operating conditions for the design of the speed reducer:

- Maximum input power
- Driver speed at rated condition
- Speed reducer ratio
- Maximum pump reverse over-speed
- Low speed shaft downward thrust including weight
- Low speed shaft upward thrust during start-up or shut-down, (if applicable)
- High speed shaft direction of rotation
- Low speed shaft speed direction of rotation
- Overhung load
- Maximum engine overload torque transmitted through the clutch
- Reverse torque load on backstop

13.3.10.3 Component Specifications

The following list of components is a generalized list and should not be considered a complete and comprehensive description of all the component pieces of a finished speed reducer. It should also be recognized, reducer designs vary from manufacturer to manufacturer and the component descriptions may not be representative of a particular design.

13.3.10.3.1 Gears

As discussed, the gearing of the reducer is the single reduction right angle spiral-bevel design. The gear teeth are precision ground or precision cut and lapped. The spiral bevel gears are gas nitrided or carburized, hardened, and lapped in pairs after heat treatment. In addition to rating the gears according to ANSI/AGMA 6010-F97 and ANSI/AGMA 2005-C96, gear stresses are specified to not exceed 80 percent of yield strength for any overload, or engine overload condition.

13.3.10.3.2 Backstop Device

A self-actuated backstop device to prevent reverse rotation of the pump due to loss of power, or drive failure, is installed as an integral part of the transmission unit. Its action is instantaneous and without backlash. The design is typically of the cam clutch type or drop-pin type and is of a capacity adequate to prevent reverse rotation with backflow through the pump due to the maximum differential pool-to-pool head. Lubrication is provided by the transmission lube oil system. The backstop is installed on the low speed output shaft. The torque is transmitted directly to the gear housing. The backstop shall operate at a temperature of less than 160 degrees F under all operating conditions.

13.3.10.3.3 Shafts

Each shaft shall be heat treated stainless steel. Welded shafts are not acceptable. Input shaft size and configuration shall be compatible with the driver. The pump head shaft shall accommodate the hollow shaft design of the reducer’s output shaft to permit vertical adjustment. Sufficient thread length shall be provided to the top of the pump shaft to permit 1-inch adjustment, either up or down of the pump shaft. The adjusting nut shall be designed to support the total axial load and thrust of the pump and be locked in position to prevent movement.

13.3.10.3.4 Seals

The down output shaft shall have a drywell design seal. The input shaft shall have a lip seal to prevent leakage of the oil and exclude dirt. Lip seals shall utilize hardened steel wear sleeves to preclude shaft repair or replacement.

13.3.10.3.5 Lubricating System

The reducer is provided with an oil lubrication system that provides continuous lubrication to the gears, bearings, and backstop. The system consists of an oil circulating pump, heat exchanger, piping, filters, and controls. Each reducer is provided its own system. The oil circulating pump is a positive displacement type pump driven from one of the reducer shafts.

13.3.10.3.6 Heat Exchanger

The maximum oil sump temperature at the rated speed and load shall be 160 degrees F at an ambient temperature of 105 degrees F. The exchanger may be either an air cooled or water cooled system. In no case, however, shall the lubricating oil piping be circulated through the water in the intake bay. If a shell and tube type lubricating oil cooler is provided, the unit shall be of adequate capacity to prevent the lubricating oil from exceeding allowable temperature limits with an entering raw water temperature of 85 degrees F.

If an oil to air exchanger is to be used, the tubes and fins shall be aluminum, copper, or copper alloy. The working pressure shall not exceed the oil pump working pressure. The exchanger shall withstand a test pressure of 150 percent of the design pressure for a period of four hours during which time the exchanger will be checked for leaks. Any leakage is cause for rejection. The oil to air heat exchanger system shall include a fan, motor, and controls to maintain the required oil temperature.

Oil to water exchanger are either water cooled shell and tube type, or water cooled plate type, or an internal water cooled coils within the reducer sump. The heat exchanger tubes are 90-10 copper nickel alloy, plates are type 316 corrosive resistant steel. The minimum wall thickness of the tubes is typically 16 gauge and designed for the pressure rating. Water shall be circulated through the tubes and plates and the design shall be such that the tubes and plates can be cleaned. The exchanger shall withstand a test pressure of 150 percent of the design pressure for a period of four hours during which time the exchanger will be checked for leaks. Any leakage is cause for rejection. The oil to water heat exchanger shall have a thermo-mechanical control valve to adjust flow rate through the exchanger to maintain a minimum oil temperature of 120 degrees F in the housing sump.

13.3.10.3.7 Piping and Fittings

Oil lines up to two inches outside diameter (o.d.) are seamless steel tubing with 37 degree flare or flareless fittings. Oil pipe equal to or larger than two inches o.d. are be black steel with welded fittings. Water piping is typically copper or copper alloy with brazed or 95-5 soldered joints. All piping, tubing, and fittings conform to ASME B31.1- Process Piping.

13.3.10.3.8 Lubricating Oil

Lubricating oil shall be mineral oil or synthetic hydrocarbon as recommended in ANSI/AGMA 6010-F97 for an ambient temperature range of 15 to 125 degrees F.

13.3.10.3.9 Oil and Breather Filters

The lubricating system shall have two oil filters on the pump outlet side. One filter shall be for removing particles and the other for water removal. Each filter shall incorporate an oil-filled differential pressure gauge to indicate the pressure drop across the filter. The filter assemblies shall be sized for a pressure drop for a clean filter of no greater than four psi. Filters shall have a bypass setting of 40 to 60 psi. Element collapse rating shall not be less than 150 psi.

- Oil Particle Filter: The Beta rating shall be B6>75 at 60 psi differential per ANSI/NFPA T3.10 1990 Filter Elements or an approved alternative. The filter shall be sized to avoid bypass at a start-up oil temperature of 80 degrees F
- Oil/Water Filter: The filter shall maintain the water content in the oil of no greater than 200 ppm
- Breather Filter: The breather filter shall have a Beta rating of B6>75 and a desiccant chamber to remove water

13.3.10.3.10 Rolling Bearings:

Rolling bearing elements are located on the shaft using shoulders, collars, or other positive locating devices and shall be retained on the shaft with an interference fit and fitted into the housing with a diametral clearance, both in accordance with the recommendations of ISO 286 (ANSI/ABMA 7 - 1995). The rolling element bearing life shall have a basic rating of L10 per ISO 281 (ANSI/ABMA 11 - 1990) of at least 100,000 hours with continuous operation at the rated condition, and at least 16,000 hours at maximum radial and axial loads and rated speed.

13.3.10.3.11 Thrust Bearings

The entire weight of the rotating element of the pump and hydraulic thrust, (up-thrust and down-thrust), imposed by the propeller and any radial loads created by the reduction gear shall be carried by the thrust bearing located in the reducer. The thrust bearing shall be sized for continuous operation under all specified conditions and shall provide full load capabilities if the pump's normal direction is reversed. The thrust bearing shall be a steep angle tapered bearing type. Misalignment of the outer and inner bearing rings shall be limited to 0.001 radian for cylindrical and tapered-roller bearings and 0.0087 radian for spherical ball bearings. Bearings shall be mounted directly on the shaft, bearing carriers are not acceptable.

13.3.10.3.12 Radial Loads

Radial load can be addressed by the thrust bearing(s) or separate rolling element bearings can be provided.

13.3.10.3.13 Housing

The reducer housing shall be cast or fabricated steel, stress relieved prior to machining, and reinforced to carry all applied loads and maintain gear alignment. The unit may be made in several sections, split as required, for service and assembly and heavily ribbed to insure strength and rigidity. The housing shall be so constructed as to provide stability that maintains precise alignment of the gears and shafts. All joints shall be finished machined and oil tight.

13.3.10.3.14 Inspection Openings

Inspection openings with cover plates shall be provided over each set of gears. All inspection, access, service and other type openings shall be provided with suitable metal covers, vented, screened and easily removable as necessary to insure continuous protection against the entrance of insects, rodents and the elements throughout the expected life of the equipment.

13.3.10.3.15 Lifting Lugs

The unit shall be provided with eye bolts or lifting lugs for installation and removal.

13.3.10.3.16 Instrumentation

The instrumentation supplied with the reducer shall be a complete working package that has been coordinated with the pump and driver supplied. The reducer shall have the following devices:

- High Oil Temperature: An oil temperature sensor shall be provided to monitor the oil temperature in the reducer sump. The alarm and shut down shall be part of the system's control and monitoring system. Lower settings may be used if recommended by reducer manufacturer. Typically the alarm is set at 180 degrees F, the shut down at 200 degrees F

- Oil Pressure: Provide a gauge after the oil pump to monitor oil pressure. The gauge shall be oil or glycerin filled and shall have an isolation value
- Temperature Gauges: Provide thermometers in the sump, in the oil line after the heat exchanger, and the backstop
- Oil Level Sight Gauge: Provide an oil level sight gauge to monitor oil levels in the sump of the reducer
- Vibration Switch: Vibration switch with the alarm and shut down shall be provided as part of the system's control and monitoring system. The manufacturer shall be responsible for the vibration switches proper settings to accommodate initial and running vibrations to avoid nuisance tripping of the switch. A time delay shall be incorporated into the control system if required. Set alarm at 0.5 inch per second or at baseline level recommended by the reducer manufacturer

13.3.10.4 Couplings

The primary function of couplings is to transmit the rotary motion and torque from the prime mover to the driven equipment. The secondary functions include:

- Accommodation of misalignment of shafts
- Transmission of axial thrust loads
- Maintain alignment between connected shafts
- Permit adjustment of shafts
- Limit torsional vibration transmission from reciprocating prime mover to driven equipment

There are two primary coupling types, rigid and flexible. Rigid couplings are necessary to maintain precise alignments or to support the position of the rotor in the drive train. An example is the line shaft couplings. There are a variety of rigid coupling types including flanged and split types.

The speed reducer to pump shaft coupling is of a rigid design and of the same material as the shaft. The speed reducer half of the coupling is typically keyed and has the proper shrinkage fit on the hollow output shaft. A service factor of 2.0 based on the maximum rated load is generally specified. In addition, the stresses shall not exceed 80 percent of the yield strength at the maximum rated load condition. The coupling shall be dynamically balanced to AGMA balance classification of 7 or better.

Flexible couplings are classified into two types, mechanically flexible and materially flexible. Mechanically flexible couplings compensate for the misalignment by clearances incorporated in the design of the coupling. Flexible couplings rely on the flexing of the coupling element to compensate for the shaft misalignment. The flexing material must have adequate resistance to fatigue failure. Typical flexing elements are composed of elastomer materials. There numerous flexible coupling types as well as elastomer materials employed in the coupling.

Definitions: Definitions shall be as defined and set forth by the American Gear Manufacturer's Association, ANSI/AGMA 1012 -F90. The following definitions apply to this report:

Efficiency The speed reducer efficiency is calculated:

η = P-output (shaft output power)/ P-input (shaft input power) x 100 percent

Reduction Ratio: The reduction gear's reduction ration at the pump's rated condition is: Input shaft rotary speed (rpm)/Output shaft rotary speed

Rating and Service Factor: The reducer is rated in accordance with AGMA 6010-F97

Torque: The formula for torque: Torque (in-pounds) = (Pp (Hp) x 63025)/rpm

Driver Horsepower: The driver horsepower is calculated: Required Driver Hp = (Torque x rpm/Gear Reduction)/(63025 x Gear Efficiency)

13.3.11 Fuel System

The fuel system design for the diesel engine drivers must conform to the requirements of National Fire Protection Association (NFPA) 30-Flammable and Combustible Liquids Code and NFPA 37-Stationary Combustion Engines and Gas Turbines. The fuel oil supply system for each engine typically consists of a motor driven fuel oil transfer pump, day tank, and an aboveground fuel storage tank(s). The fuel oil flows through a strainer from the outside fuel oil storage tanks to the day tank. From the day tank, oil flows to the engine. The overflow and drip lines from the engine return the oil to the day tank. If the main storage tank is the lowest point in the engine fuel system, a pump may need to be provided to deliver fuel from the tank to the day tank.

13.3.11.1 Primary Storage Tanks

Unless there is an over-riding reason, such as lack of adequate site area, the primary storage tank(s) should be aboveground double walled tanks. The tanks are UL 142 listed and normally store the specified petroleum product at atmospheric pressure but should also be designed to withstand a pneumatic pressure test. The secondary containment structure shall also be UL 142 listed. The number of tanks can depend on a number of criteria. All tanks should be of equal capacity. A minimum of two tanks is preferred to allow for out of service maintenance. Capital cost is typically the over-riding factor in the determination of the number of tanks. However, space restrictions and foundation problems can play a role in the determination. The tank(s) nominal capacity should provide for a minimum of ten days of fuel for continuous operation of all units operating at maximum horsepower. This criteria typically applies to remote stations or situations, such as hurricane events, where fuel deliveries would be severely disrupted. Reduced capacities can be specified for less critical applications or where the owner has confidence in dependability of fuel deliveries.

The primary tank is fabricated from steel and is a welded construction throughout. The fabrication is in accordance with UL-142 with steel conforming to ASTM materials, grades and thickness. All welded joints of the tanks must have full penetration and complete fusion. All welds are subjected to a soap film test, using a vacuum device or other approved method. A factory pneumatic pressure test is typically specified. The air pressure test should be applied in the manufacturer's shop at five psig held for a period of two hours without a pressure drop after the test apparatus has been removed.

The secondary containment shall be a second steel wall. All tank openings shall be located on top of the tank. Catwalks shall be provided on top of the tanks to permit access to all tank openings and piping connections.

The tank openings and piping connections that are recommended to be furnished:

- Emergency vent
- Tanker fill
- Vent
- Level gauge
- Overfill containment
- Fuel oil supply
- Fuel Oil Return
- Remote fuel inventory

Storage tank accessories that are recommended to be specified:

- Lockable fill cap
- Vent cover with 40-mesh screen over the outlet, and an aluminum cover to prevent rain from entering vent line
- Emergency vent to relieve internal pressures in excess of 2.5 psi. The vent shall be sized according to NFPA 30 requirements

After shipment to the job site, the tank is pressure tested at psi air pressure for one hour and checked for leaks, using a soap solution at all connections. During testing, connections may be plugged but shall not be blocked or plugged on the inside. Any leaks or indications of leaks in the tank or accessory connections are repaired and made completely airtight. A suitable pressure relief valve is used during the test to prevent over pressurization of the primary tank.

13.3.11.2 Day Tank

The day tank is a unit composed of a small capacity fuel storage tank with secondary containment, fuel transfer pumps, and level controls and is located inside the pump house near the engine driver. The unit serves to transfer fuel to the engine from the outside storage tank at a controlled suction head and delivery rate. It also functions as a collection point for transfer of the return fuel to the outside tank. A day tank is furnished for each engine. The tanks are of steel construction, double walled with leak detection monitoring, and built in accordance with the applicable provisions of the NFPA 30 and UL 142. A motor driven fuel oil positive displacement type pump, with built-in relief valve and capacity as recommended by the engine manufacturer, is typically furnished with each day tank. The pump transfers fuel oil from the storage tank to the day tank. The engine's fuel pump transfers the fuel oil from the day tank to the engine. A line is provided for return of unused fuel to the day tank and motor driven fuel oil positive displacement type pump is provided for return of overflow fuel back to the storage tank. Both fuel transfer pump assemblies should be included as part of the day tank package. The capacity and performance criteria of the day tank unit specified shall be verified by the designer to ensure proper performance of the engine supplied. In addition, the designer should confirm the piping distance from the engine to the day tank is acceptable to the engine manufacturer. Typically the fuel storage capacity of the day tank is based on the fuel return rate and the volume of fuel needed to ensure proper cooling of the fuel in accordance with the requirements of the engine. The total storage capacity of all the day tanks located inside the pump station shall not exceed 1,320 gallons.

The day tank highest fuel level shall be located below the engine injector to prevent run-on. The day tank’s lowest fuel level needs to be above the engine-driven fuel pump to ensure the pump maintains its prime. The day tank should be completely factory assembled, wired, painted and tested. The following additional features are typically specified:

- Ports for supply, inflow, fill, and overflow lines
- Vent with flame arrestor connections. The vent shall exhaust to the building exterior and at an elevation of five feet above the top of the tank
- Level Indicator, side mounted, direct reading float controlled liquid level indicator
- Liquid level switch for automatic control of the fuel oil transfer pump and high and low level alarms
- Drain with shut-off valve

The tank should be all seam welded, square, atmospheric tank of heavy gauge steel with internal reinforcements and pressure tested to five psi test pressure. The tank should be provided with welded flange pipe fittings for overflow, vent, and drain lines. All fittings except drain located above normal full level. The unit should be mounted on heavy gauge steel channel feet with mounting holes. The tank should also have a removable steel top cover. The tank requires an overflow basin for containment of 150 percent of the tank capacity. The basin needs to circle the tank and include a drain. The unit should have corrosion resistant interior and exterior finishes.

13.3.11.3 Transfer Pumps

The fuel transfer pump system includes high vacuum, single stage, internal-gear, positive-displacement rotary type pumps of non-corrosive alloy and carbon composition with leak-proof mechanical rotary shaft seal. Each pump is driven by a 120V, single-phase open drip proof (ODP) motor with thermal overload protection. Each pump is protected by a pressure relief valve appropriately sized for the pump provided. The fuel oil transfer pump shall be sized to provide 150 percent of the combined fuel consumption rate.

13.3.11.4 Fuel Inlet Equipment

The fuel inlet equipment includes

- Fuel strainer
- Priming tee and check valve
- Solenoid valve, 120V
- Foot valve shall be installed at supply line termination in main storage tank

13.3.11.5 Controls

The day tank controls and features can vary from model to model. The following list is typical of the requirements specified:

- Pump run-off-automatic operation
- Press to test pump pushbutton
- Pump start-stop automatic control. Level control switch shall be industrial quality mechanical float switch with double pole double throw (DPDT), 2-Hp contacts, welded steel balanced float, and adjustable pick-up to drop-out differential.

- Local/remote low fuel level alarm consisting of red alarm light plus dry signal contacts for remote alarm. Activated by separate float switch sensing at 30 percent of day tank fuel capacity.
- Local/remote high fuel alarm consisting of red alarm light plus dry signal contacts for remote alarm. Activated by separate float switch sensing 95 percent of day tank fuel capacity.
- High fuel level emergency pump stop switch to override main float switch and stop pump motor at 95 percent of day tank capacity.
- Mechanical float gauge
- Indicators shall be long life, bright, large display light emitting diodes (LEDs) and shall include the following indication functions:
  - Fuel level
  - Power available
  - Switch off (flashing)
  - Pump running for each pump in duplex package
  - Low level alarm
  - Overflow alarm/pump control backup activated
- Supply the following outputs
  - Pump start-stop
  - Low level alarm
  - High level alarm
- Low Level Float Switch: Provide low level float switch for engine shut down to prevent starving of injectors and need to re-prime the engine. Engine shut down shall be shall be activated by engine control system in accordance with normal shut down procedures.

13.3.11.6 Installation

To ensure the safety controls work properly, a high level alarm test shall be performed. The day tank shall be manually filled to a level above the overfill limit. The level that activates the alarm shall be recorded and the shutdown of the fuel transfer pump shall be verified. The day tank shall be drained below the overfill limit following the test. In a similar manner a low fuel alarm test shall be performed. Fuel from the day tank shall be drained to lower the fuel level below the no fuel level limit to test the audible alarm.

13.3.11.7 Fuel Piping and Auxiliaries

The following piping and auxiliaries are part of the fuel system:

13.3.11.7.1 Fuel Filters

Each engine supply line shall have a duplex filter with valve installed on the inlet side of the engine fuel pump. The filter shall have the capablity of filtering out particles down to 25 micron size.

13.3.11.7.2 Fuel Strainers

A full flow fuel strainer is be provided in the fuel oil system upstream of the engine and duplex filters. The strainer shall be a replaceable cartridge type capable of filtering out particles down to 125 micron size.

13.3.11.7.3 Fuel Meter

A rotating disc type fuel meter is furnished for measuring fuel oil supplied to each day tank. The meter is especially designed for this service and calibrated in U.S. gallons, with a five place cyclometer dial. The meter is located in the system to measure net engine fuel consumption.

13.3.11.7.4 Single-wall Piping

Single-wall piping is required to meet the standards set forth in ANSI/ASME B36.10. Pipe shall conform to ASTM A53 Grade B, Schedule 40, seamless or electric resistance. No pipe or fittings in the piping systems should be galvanized. Fittings for screwed pipe are typically specified as 3,000-pound forged steel conforming to ANSI/ASME B16.11.Flanges shall be standard weld-neck type 150-pound forged steel, ASTM A-181, and conforming to ANSI/ASME B16.5.Flange facings shall correspond to the equipment to which the piping is joined, and, unless otherwise required shall be standard 1/16-inch raised face flanges. Machine bolts are heavy hexagonal alloy steel conforming to ASTM A307, Grade B. Nuts shall be heavy hexagon alloy steel conforming to ASTM A563, Grade A. All flexible oil lines, such as connections to the engines, should be specified as reinforced nitrile hydraulic hose with stainless steel braided sheathing.

13.3.11.7.5 Double Containment Steel Fuel System Piping

Double wall piping consists of a steel carrier pipe within a steel containment pipe. The internal carrier piping is typically standard weight carbon steel, ASTM A53, Grade B pipe. All carrier pipe joints are butt-welded for 2.5 inches and greater, and socket welded for 2 inches and below. Carrier pipe fittings are carbon steel butt weld or socket weld fittings. The secondary containment pipe is fabricated out of ASTM A-139B, Grade B, ASTM A-120, Grade B or ASTM A53, Grade B carbon steel, Schedule 40 for pipe diameters less than six inches, and schedule 10 for six-inch diameter and above. Joints of secondary containment pipe are butt-welded with carbon steel butt weld fittings. The carrier pipe inside the containment casing is supported at 10-foot intervals or less. The supports are designed to allow for continuous air flow and drainage. The support spacing is dependent on the pipe diameter. Carrier pipe and containment pipe are required to be air tested.

Double containment piping shall be used exterior to the station in areas where spill protection is required such the supply and return lines to the fuel storage tanks.

13.3.11.7.6 Pipe Hangers and Supports

Pipe support or hanger spacing and arrangements should conform to ANSI/ASME B31.1 Code for Pressure Piping. Pipe supports or hangers are provided as required and at changes in pipe direction to limit pipe deflection under the applied load and suppress vibration. The complete hanger assemblies need to be adequately rated for the applied load and be designed for potential expansion. Pipe hanger and supports shall be of the types listed in Table 1 "Hanger and Support Selection," MSS Standard Practice SP-69 except that the following figure types given in Figure 1 are not acceptable: Types 5, 6, 11, 12, 7, 9, 10, 16, 17, 23, 20 and 25.

Install fuel piping systems in accordance with NFPA 30 (take special note of Chapters 2 and 3), NFPA 30A (take special note of Chapter 4 and 8), local codes, manufacturer's requirements for warranty and latest EPA and state regulations for fuel storage tank systems. The following installation requirements should be followed:

- Install each run with a minimum of joints and couplings, but with adequate and accessible unions for disassembly and maintenance or replacement of valves and equipment
- Reduce sizes (where indicated) by use of reducing fittings
- Align pipe accurately at connections within 1/16-inch misalignment tolerance
- Comply with ANSI/ASME B31.1 - Code for Pressure Piping
- Locate piping runs, vertically and horizontally (pitched to drain)
- Orient horizontal runs parallel with walls and column lines
- Hold piping close to walls, overhead construction, columns and other structural and permanent-enclosure elements of the building
- Thread pipe in accordance with ANSI/ASME B1.20.1
- Welding shall be accomplished by the use of the shielded metallic arc process and shall be in strict accordance with ANSI/ASME B31.1
- Butt welding end preparation on all pipe shall conform to ANSI/ASME B16.25
- Provide sleeves for all openings in walls required for pipes and tubing
- Paint all exposed steel piping

13.3.11.7.7 Pneumatic Pressure Tests

After installation of tank and piping is complete the pressure test of the piping system including tank connections, fittings and piping is performed.

- Apply a preliminary test at 25 psig. Final test pressure shall be as tabulated
- Provide a means of bringing the systems up to a higher pressure as tabulated
- Perform in accordance with paragraph 137 of ANSI/ASME B31.1 for all pipelines
- Maintain test pressure for at least four hours
- Pressure and temperature readings shall be taken
- Temperatures shall be representative of actual conditions

- Readings shall not be taken during times of rapid atmospheric changes
- There shall be no indication of reduction in test pressure after corrections for temperature and pressure have been made according to the relationship $T_1P_2 = T_2P_1$ where T and P are absolute temperatures and pressures and subscripts refer to initial and final readings.

13.3.11.7.8 Precision test

After tank and piping has been installed and filled a precision tightness test as defined by NFPA 329-4.3.10 as a final test for leaks in the system is performed.

The test results shall show tank and piping tightness conforming to EPA regulations as described in 40 CFR Part 280.41 (c) (1) (less than 0.1 gallon per hour leakage). Should the tank system fail the test, the Contractor shall take immediate action to repair any leaks and shall be liable for all charges incident to such repair and the clean up or any resulting contamination.

13.3.12 Trash Racks and Rakes

The trash rake and conveyance system is custom engineered equipment that is complete with all structural support, motors, and monitoring and control components for the removal trash accumulated on the trash rack and its conveyance to a disposal area. There are a wide variety of systems available, unique in their design and operation. The equipment may be supplied independent or in combination with the rack. The type of raking device used to remove trash from the rack depends on the estimated trash type and loading rate, the environmental conditions of the region, the size of the station and in some cases the space available. In the past, raking for small stations was done by hand. However, new installations are typically equipped with automated rakes that collect the trash and transport it to a dump site.

13.3.12.1 Rake Types

There are a number of trash rake designs, the following summary includes the common types available.

13.3.12.1.1 Chain Driven Scrappers/Rakes

There is a variety of the front cleaning scraper bar type trash rakes. This rake type uses horizontal scraping bars supported by chains that rotate on sprockets driven by an electric motor. The bars clean from the bottom to top in front, riding on the rack either returning on the downstream side of the rack or over the raking bars. The scraping bars are typically made of polyethylene materials of varying widths and penetrate between the rack bars. Debris is piled behind the rack on the service bridge and in conveyed to the disposal site via a conveyor or by other methods such as a loader. The operation is continuous with the operation of the pump. All submerged materials of the rake are made of corrosive resistant materials. The unit is easily adapted to different site conditions, intake geometry, and type of debris by varying the chain and scraping bar length. The speed of the rake can be varied to adjust to the estimated trash loading. The unit is limited to 1,000 pounds items and excessive amounts of debris can jam the chain drive. Maximum rake widths are 12 feet requiring multiple units to service intake widths of greater dimensions. There are a number of advantages of this rake in addition to its continuous raking operation. The unit can be easily installed and can be adapted to an existing installation. The unit can also operate effectively in high wind and is not vulnerable to flying debris.

13.3.12.1.2 Hydraulic Boom Grippers/Rakes

The hydraulic boom type rake consists of two piece boom that is driven by hydraulic cylinders in a similar manner as a backhoe. The boom is mounted on a trolley that rides on crane rails mounted on the service bridge. Other styles for smaller intake widths pivot on a stationary pedestal and dump their trash to the side, again in a similar manner as a backhoe operation. There are variations of this hydraulic telescoping boom style rake, one is a single piece arm that moves up or down tilting the head unit as the boom re-tracts into the head as it rakes upward.

The monorail or track mounted style rakes can cover intake widths of 30 feet or more with a single unit. The raking speed is up to 45 feet per minute. Therefore to clean a 12 foot deep trash rack, the cycle time will be approximately 40 seconds plus the dumping time, a total of about one minute.

There are a number of styles of rakes or grippers that hydraulically open and close. The rake style is positioned by the boom below the debris and is raked along the rack moving the trash up and onto the service bridge or into a bin. The gripper style descends in an open position with the rear tooth penetration through the rack bars. The descending rake head captures the debris on the way down and closes grabbing the trash in a similar manner as a clam shell excavator. The gripper then dumps into a bin with the entire unit traveling on crane rails along the service bridge. The gripper style rake has the advantage of being able to handle large trash size with a capacity of 3,000 pounds.

13.3.12.1.3 Cable Suspended Gripper/Rake

The cable suspended style rake consists of a rake or gripper that is lower by cables and dragged along the rack to collect the trash is a similar manner as a dragline excavator. The rake style moves the trash up the rack and dumps into a bin in the mobile carriage that rides on crane rails on the service bridge. Another style uses a gripper that closes on the trash and lifts by an electric motor driven unit suspended from a monorail that travels to the dump site.

13.3.12.1.4 Guided Cable Hoist Rake

Guided cable trash rakes cover the full width of the rack and have fixed channels on each side of the intake bay to guide the rake’s travel. The rake is operated by a cable hoist system. The rake is lowered by gravity and raised by the hoist machinery. The teeth of the rake are designed to pull away from the rack when lowered and engage the rack when raised. The width of the rake is typically limited to 15 feet and the vertical travel limited to 40 feet. The rake can be designed to run on rails to service multiple bays or can be fixed at one location. There are unguided types of cable hoist rakes that are similar in design.

13.3.12.2 Design Considerations

The various models of trash rakes are unique in their design and operation and are difficult to compare. An important selection consideration is the performance of the rake under storm conditions. In hurricane prone regions, operation during high wind events can be the governing criteria. Where wind is not the over riding consideration the rake's ability to remove heavy trash loads or large floating items may be the deciding factor. The rakes operation with ice may be an important factor in the cold climates.

13.3.12.2.1 Operating Conditions

The rake and conveyance equipment shall be suitable for outdoor service. The hydraulic system design of the station assumed a one foot maximum head loss across the trash rack. The cycle time and the load lifting capabilities of the rake proposed to be supplied needs to be adequate to prevent the build-up of trash on the rack that will cause a head loss in excess of one foot. The conveyance equipment should also have the capacity to move the collected trash to the dump site at a rate equal to or better than the collection rate of the rake(s).

13.3.12.2.2 Trash Loading

An estimate of the loading rate will depend on countless factors. It can be assumed that during a storm event anything can end up in the waterway. For example, during Hurricane Andrew, in addition to numerous dead livestock being blown into the canal, a pick-up truck was lifted and dropped on the side slope of an intake channel. These situations can not be addressed beforehand but there is a need to specify loading criteria for the rake to ensure the equipment supplied has adequate capabilities for the more severe conditions. It may not be a storm event that causes these heavy loads. Constructed wetlands after start-up of the discharge pumps after a dormant period can up-root the new growth vegetation causing an extreme slug of trash.

Trash loading is also not a steady state event. Typically the maximum loading occurs at start up of the pump and diminishes with time. However, given the pump is continuously operating during a storm event, there is the possibility the maximum loading may occur as a result of an upstream situation caused by the storm. One method to specify the estimated loading rate is by a percentage of the pump water volume. This estimate may be more useful for sizing the dump area than the rake. A reconnaissance of the tributary area of the pump should be made to identify the type of debris that can be expected and develop an idea of the potential amount that could be occur during a high water flood and/or wind event. Historic records and the experiences of the operators are the best source to for specifying the estimated debris type and loading rate.

Constructed wetlands produce a considerable amount of vegetative trash. Under normal operation the trash will consist of submerged and floating vegetation consisting of aquatic and wetland vegetation. This vegetation may vary from tall grasses, reeds and other aquatic plants such as hydrilla to floating leafy plants such as water hyacinth. The rate of loading will vary seasonally as well as with discharge conditions. During a flood event there is the possibility of larger debris being part of the trash load. This debris could be upland vegetation including small trees and brush as well as man-made items such a lumber.

13.3.12.2.3 Handling Capacity

The rake and its conveyance equipment should be capable of collecting and conveying the debris specified in a cycle time that results in a head loss across the rack less than the maximum allowed. Because of the uncertainty in the rate of trash loading, the cycle time needs to be capable of adjustment to optimize the rake's performance. The rake manufacturer should visit the project site and make an evaluation of the service condition requirements to determine the necessary handling capacity of the rake/conveyance equipment proposed to be supplied. The weight of this trash can be assumed to be 35 pounds per cubic foot. The minimum handling capacity per rake should not be less than 1,000 pounds.

13.3.12.2.4 Standby Service

The rake/conveyance equipment shall be designed for standby service which is defined as a normally idle piece of equipment that is capable of immediate automatic or manual start-up and continuous operation. The rake should operate when the pump is operating. The engineer should provide an estimate of the annual pump operating time.

13.3.12.2.5 Intake Design

The trash rake and conveyance equipment are mounted on the service bridge. The equipment is positioned on the intake side of the bridge. The layout of the equipment should ensure the vehicular traffic across the bridge is not impacted by the system. The rack inclination is dependent on the type of rake employed. The flatter slope of the rack, typically the greater the trash load the rake can remove. The flow velocity into the intake also impacts the performance of the rake.

13.3.12.2.6 Removal and Re-installation

The rake and the trash rack must be capable of being removed for access to the pump. The rake should require the minimal dismantling necessary to facilitate this maintenance requirement and needs to be designed for easy re-installation.

13.3.12.2.7 Maintenance

Submerged components shall not require service, lubrication, etc. i.e. bearings. The layout of the trash conveyance equipment shall allow for easy access to the major operating parts of the mechanism for maintenance, cleaning and repair.

13.3.12.2.8 Operation Control

The operation of the rake and conveyance equipment shall be determined by the position of an "Auto–Off–Manual" selector switch located on the Control Panel. "Auto" mode shall interlock the rake and conveyance equipment with the operation of the pump and shall require no operator interface for proper operation of the system. "Manual" mode shall allow operation of the system or any of the system components by an operation.

13.3.12.2.9 Electrical Controls

The electrical control panel for the rake/conveyance equipment are located in the pump station building. Local controls, disconnects, etc. as required, should be located adjacent to rake equipment on the service bridge. The operation of the trash conveyance equipment if integral with or a separate mechanism from the trash rake is controlled by the control panel. If the trash conveyance equipment is a separate mechanism from the trash rake the operation of the two systems shall be interlocked.

13.3.12.2.10 Load Limiting Device

The rake shall be provided with a load limiting device that shall shut down the rake and signal an alarm condition back to the control panel, if the pull force on the rake exceeds the setting of the load limiting device.

13.3.12.2.11 Structural Design

The design of the structural support and anchorage of the rake and the conveyance equipment shall be in accordance with the AISC Manual of Steel Construction. Welding shall be in accordance with ANSI/AWS D1.1 and AWS D1.6 – Structural Welding Code.

13.3.12.2.12 Wind Loading

The rake and associated conveyance equipment shall be designed to withstand without structural damage sustained wind speeds up to 155 mph. The rake/conveyance equipment shall also be designed to operate without loss of effectiveness up to sustained wind speeds of 70 mph. The rake manufacturer shall provide a written certification the unit proposed for use is capable of satisfying the wind load requirements specified above.

13.3.12.2.13 Safety

The equipment shall comply with the applicable provisions of the OSHA.

13.3.12.2.14 Materials of Construction

All components subject to submergence shall be fabricated from a corrosive resistant material.

13.3.13 Discharge Piping and Appurtenances

For the smaller pump diameters without a formed discharge tunnel, each pump is connected to a steel discharge pipe of and equal diameter to the pump barrel. A flexible compression coupling is typically used to connect the pump elbow to the discharge line. Flexible couplings should be used whenever the pipe runs into or out of concrete structures or any other location where differential settlement is anticipated.

The pump discharge piping shall be designed to withstand all stresses resulting from external loads and internal pressures. The pipe shall be fabricated to accommodate thrust rods or other means of constraint as specified. Flanged wall thimbles shall be provided for all penetrations through structure walls. Flanged connections shall be provided for the discharge pipe and flap valves.

13.3.13.1 Pipe

The pipe design shall conform to AWWA C200, AWWA M11, and as specified except that hydrostatic test of fittings after fabrication will not be required.

Wall thickness shall be tabulated hereafter:

Table 13.3-15 Pipe Wall Thickness

Normal Pipe Diameter (inches)	**Thickness** (inches)
<48 inches	3/8 inch
48-inch to 60-inch	1/2 inch
60-inch to <84 inch	5/8 inch

13.3.13.1.1 Materials

The pipe shall be fabricated from one of the following materials:

- Sheet or coil conforming to the requirements of ASTM A570, Grades 30, 33, 36, or 40
- Plate in coil form conforming to the requirements of A36, A283, Grades C or D, or A572, Grade 42
- Coil conforming to the requirements of ASTM A-139, Grades A or B

13.3.13.1.2 Joints

Provide flanged joints shall be specified for locations. All joints shall conform to AWWA C200 and AWWA C207 and the flanges shall have a pressure rating AWWA Class B and be drilled ANSI B16.1 Class 25.

13.3.13.2 Fittings and Special Connections

Elbows shall be fabricated from tested pipe to conform to AWWA C208 and shall be reinforced in accordance with applicable provisions of AWWA M11. Openings for air vent connections shall be provided with flanged outlets and shall be flanged in accordance with ANSI/ASME B16.5 standard 125 pound flange.

13.3.13.2.1 Harnessed Coupling

A flexible mechanical coupling, Dresser style or equal, shall be provided to connect the pump discharge elbow to the discharge piping. All components of the coupling shall be stainless steel. The connecting ends of the discharge pipe shall be fabricated in accordance with the requirements of the coupling provided. Adjustable thrust rods shall be provided to transfer thrust loads to the discharge piping or wall thimble. All bolts, rods, nuts, and associated hardware shall conform to ASTM F593 Type 316 stainless steel.

13.3.13.2.2 Wall Thimble

A wall thimble shall be provided for embedment in the intake back wall and connection to the pump discharge elbow and the discharge piping or flap valve. The thimble shall have a seal ring centered in wall when embedded and shall have flanged ends to mate to the discharge piping.

13.3.13.3 Gaskets and Bolting Materials

Gaskets for flanged joints shall conform to ANSI B16.21, 1/8-inch thick full-face synthetic rubber. Full-face gaskets for all pump and equipment connections shall be provided. Bolts for flanged joints shall conform to ASTM F593 Type 316 stainless steel. Nut and bolt heads shall be hexagonal.

13.3.14 Lubrication Oil System

A lube oil system suitable for unloading, storage, and transfer of supply and waste lube oil will be provided, including all necessary storage tanks, pumps, piping, valves, controls, and accessories. Lube oil and waste lube oil storage systems will have a minimum capacity of 30 days storage, based on equipment manufacturer recommended oil change capacity and intervals.

Storage tanks will be aboveground single wall type and will be designed and constructed in accordance with applicable industry codes, including API and UL. Tanks will be provided with level detection and overflow prevention devices. The system will be designed for truck unloading of lube oil and loading of lube oil waste, to include all necessary storage tanks, pumps, piping, valves, and accessories for unloading lube oil and loading lube oil waste. System design will facilitate minimal loading and unloading time. Lube oil and waste lube oil pumps will be self-priming, positive displacement type. Pumps will be motor-driven and equipped with an integral internal relief valve. Lube oil piping will be ASTM A53 or A-106 black steel piping. Minimum pipe wall thickness will be based on ASME B31.3. Lube oil piping and fittings will be butt welded or socket welded. Butt weld fittings will be in accordance with ASME B16.9 and socket weld fittings will be in accordance with ASME B16.11. Valve construction and class will be in accordance with ASTM B16.34. Underground piping will secondarily contained in a fiberglass reinforced plastic containment system. Containment piping will be capable of withstanding H-20 highway loading, as defined by AASHTO HB-16.

13.3.15 Vacuum Priming System

The vacuum system for each station will consist of two electric-powered pumps, one on-line and one standby, to remove air from the pump discharge pipe to establish full flow through the pump discharge. To establish vacuum in each inflow pump discharge conduit, air will be drawn out through two eight-inch to 12-inch ports from the discharge tube which will then be manifolded together and directed to a barometric tube and then to a vacuum pump. Each vacuum line will have its respective vacuum release valves, with both auto and manual operation, providing two vent areas for siphon break to minimize time to break siphon on shut down. The vacuum pumps, which will alternate, will be manually started from each engine control panel and will utilize a barometric tube separator between the vacuum pump and the main pumps to protect the vacuum pump from water slugs. The system will signal run status to the control console. A sensor will monitor vacuum pressure and send a high vacuum alarm to the control console. Seal water for the vacuum pumps, if required, will be supplied by the cooling water system and will have solenoid controls. Selection of pump size should be based upon an eight to 12 minute time to evacuate all air from submerged suction and discharge tubes.

13.3.16 Compressed Air System

Given the decision to air start the engines, the compressed air system shall consist of two air compressors, (one redundant), with a starting air receiving tank for each engine and one air receiving tank for the instrument and service air. The compressed air system will include all necessary equipment and accessories, including compressors, receivers, dryers, filters, motors, piping, instrumentation, and controls. The air compressor will be air cooled and oil lubricated. The compressor will be motor-driven with the motor and compressor mounted on a fixed base. The compressor will utilize a belt drive with a spring-loaded tensioner. Discharge pressure will be as required for engine starting with a minimum pressure of 150 psig. Accessories will include oil filter, air filter, silencers, and vibration isolators. A receiver will be provided for instrument and service air. Receivers will be ASME code stamped vessels and will include a pressure relief valve and automatic condensate drain. Pressure regulators shall be provided for instrument air and service air. Refrigerated dryer for instrument air suitable for continuous operation shall be provided. Dryer will be equipped with inlet and outlet filters and an automatic condensate drain.

Compressed air piping will be ASTM A53 or A-106 black steel piping. Minimum pipe wall thickness will be based on ASME B31.3. Fittings will be in accordance with ASME B16.9 and B16.11. Valve construction and class will be in accordance with ASTM B16.34.

13.3.17 Backflow and Dewatering Gates and Operators

The general application rule dictates that for structure bay widths of 10 feet or less a single stem fabricated stainless steel slide gate shall be specified. For widths over 10 feet, a twin stem gate operator or drum and cable hoist and roller gates shall be specified. The gate and operator specifications shall written to ensure a reliable, durable, and low maintenance water control gate of high industry standards. The gates are to be manually or automatically operated. However, operation can be relatively infrequent so the gate design must consider the reliability of operation after periods of inactivity of a year or more.

13.3.17.1 Gate Frames

The gate frame shall be embedded into the reinforced concrete of the structure.. The frame including the guide rails, mounting frame, sill, and yoke shall be a rigid, welded unit extending from the floor of the structure to the operating platform.

The yoke or horizontal operator support member shall satisfy the following criteria:

- The yoke shall be designed to provide the required structural support for the operator and the loads produced from the gate operation under maximum head differential conditions
- The structural members of the yoke shall be sized to limit deflections to 1/360 of its span and prevent harmful rotation
- The yoke shall have a minimum strength of not less than twice the rated thrust output of the operator
- The yoke shall allow removal of the gate without disassembly

Guides shall satisfy the following criteria:

- Guides to run the full length of the disc
- Capable of taking the total thrust produced by water pressure
- Capable of supporting the disc when the gate is open

13.3.17.2 Seals

The gate seals shall satisfy the following criteria:

- A top gate seal shall be provided to prevent leakage over top of gate when closed.
- Seals shall be securely fastened to the slides and sill using dovetail groves or other methods which securely seat seal in place.
- The seals shall be easily removed and replaced for maintenance.
- The seal shall be adjustable to ensure full contact with gate disc to limit leakage through the full range of stage.

13.3.17.3 Disc

The slide cover shall be a flat plate, reinforced with structural members welded to the plate as required for the application. The disc shall satisfy the following criteria:

- The structural design of the disc shall limit deflection to not more than 1/360 of the span under the maximum loading condition
- The stem connection shall be of a standard design with a thrust nut supported in a welded nut pocket. The pocket shall have a minimum design strength of not less than twice rated thrust output of the operator
- The disc shall be designed to drain and not trap debris, soil, etc.
- Provide a drain for thrust nut assembly

13.3.17.4 Stem and Thrust Nut

The lift stem shall satisfy the following criteria:

- Thrust nut designed for all loads during opening and closing
- Thrust nut locked to prevent turning in the slide
- Stem design force shall not be less than 1.25 times the output thrust of the electric motor lift unit in the stalled motor condition
- Stem threads shall be machine cut or rolled and of the square or Acme type. The number of threads per inch shall be as required for efficient operation in accordance with the lift mechanism used
- Provide rising stem with adjustable stem guides to ensure alignment and stem support
- Provide a stop collar at the full-open position
- Each rising-stem unit shall be provided with a stem cover
- The cover shall be of sufficient diameter and length to permit full travel of the threaded stem without obstruction
- The top of the stem cover shall be closed
- The bottom end of the stem cover shall be vented, drained, and mounted in a housing or adaptor plate for easy field mounting installation
- Indicator. Each actuator for rising-stem gates with a galvanized pipe cover shall be provided with a position indicator to show the position of the gate at all times. The indicator shall be attached to the mechanism

13.3.17.5 Electric Operators

The lifts shall have electric motor operators with rising stems and stem covers. The gates shall have manual/auto operation. A factory-mounted on pedestal type operator shall be specified. The driver shall be an ac electric motor operator with stop-open-close operation on 208 Vac, 3 phase service, 60 Hz, with position indicator. The electric operator shall open and close the gate at a nominal speed of six inches per minute. The pedestal height shall be approximately three feet. An auxiliary manual crank shall be specified for hand wheel operation of lift. The crank shall be removable and fitted with a corrosion-resistant rotating handle. The maximum crank radius shall be 15 inches.

13.3.17.6 Lift Mechanism

The lift mechanism shall be supplied with a pedestal, machined and drilled to receive the gear housing, and drilled for bolting to the operating floor. The mechanism shall be geared to limit the slide operation to not more than 40 foot-pounds on the lifting device after the slide is unseated from its wedges. A lift nut threaded to fit the operating stem shall be provided. Ball or roller bearings shall be provided above and below the flange on the lift nut to take the thrust developed in opening and closing the gate with a force of 100 foot-pounds (135 N-m). Gears shall be machined accurately with cut teeth to provide smooth, proper operation for the lifting mechanism. Suitable shafts shall be installed with sleeve, ball, or roller bearings of appropriate size. All gears and bearings shall be enclosed in a housing. Fittings shall be provided so that all gears and bearings can be lubricated periodically. All geared lifts shall be suitable for operation by use of a portable-motor apparatus. The lift shall be self locking at any position of the stem travel.

The lift shall satisfy the following criteria:

- Furnish the following limit switches mounted in the lift assembly for control of the electric motor
- Provide adjustable torque responsive switch
- Provide intermittent adjustable gear type limit switches for stop of gate travel
- Opening direction. The direction of wheel or crank rotation to open the gate shall be indicated on the lift cover
- Lubrication: Provide totally enclosed gearing with lubrication fittings and all necessary mechanical seals

13.3.17.7 Electric Motor

The motor shall be specifically designed for the electric actuator service, with a high starting torque. The motor horsepower shall be determined by the manufacturer.

13.3.17.8 Materials

The materials specified are considered the minimum acceptable for the purposes of durability, strength and resistance to erosion, and corrosion. The manufacturer may propose alternative materials for the purpose of providing greater strength or to meet required stress limitations. However, alternative materials must provide at least the same qualities as those specified for the purpose. All material shall conform to the latest ASTM specification or other listed commercial specifications covering the class or kind of material to be used.

13.3.18 Station Emergency Power

Given a power outage the flood control station requires emergency electric power backup. Typically a adequately sized engine generator is provided for back up station service during the time utility power is lost. In large flood control station a second redundant system is provided. Controls are provided in the 125 ampere ATS to either manually or automatically start the engine-generator. A RTU output command will remotely STOP the engine generator should the unit unnecessarily start.

13.3.19 Stage Monitors

Upstream and downstream stilling wells with level transmitters provide analog water level data of the approach and discharge channels. A stilling well with a level transmitter is also provided in each pump intake to monitor low water levels. The water level transmitters provide a proportional 4 – 20 ma signal. The stilling well level transmitters are normally continuously powered from a 120 VAC to 24 VDC power supply. Upon the loss of 120 VAC service, the RTU's "PULSE ANALOG CIRCUITS" program will intermittently power and scan the analog water level transmitters.

13.4 *CONTROL/GATE STRUCTURES*

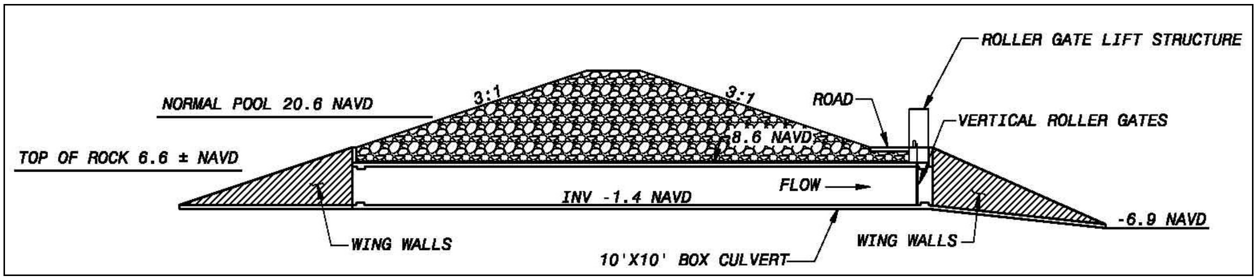
Gate structures are required to control the release of water from the EAA Reservoir A-1 to the NNRC and the STA-3/4 Supply Canal, and to control the flow of water from the STA-3/4 Supply Canal to the EAA Reservoir A-1 when the G-370 and G-372 pump station are discharging into the EAA Reservoir A-1. The specific hydraulic requirements of the gate structures at the EAA Reservoir A-1 site are discussed in Section 6.6.

As discussed in the Gate Evaluation Technical Memorandum, vertical roller lift gates are recommended due to the SFWMD's familiarity with the operations and maintenance of these types of gates.

In general, these gate structures will conform to the gate configuration shown in the SFWMD Design Guidelines Sheet G-S1. This standard shows the general criteria for the SFWMD standard water control structure with vertical roller lift gates. The structures will each have a precast control building. An ogee weir, or in the case where flow is bi-directional, a trapezoidal weir, and the vertical roller gates will maintain the desired water surface elevation. The operating platforms will be elevated above the structure walls as necessary for proper gate operation. Each structure will also have a service bridge and dewatering capability to enable dry maintenance. The service bridge will carry maintenance roads serving the EAA Reservoir A-1 embankment and the STA-3/4 Supply Canal where required.

The culvert/gate structures being considered for the EAA Reservoir A-1 are 10 feet x 10 feet box culverts with vertical roller gates located on the external embankment. They will be submerged structures, with the invert at -1.6 feet NAVD88, or 10 feet below the surrounding ground surface. To achieve the appropriate gate width, parallel 10 feet x 10 feet culverts will be placed in series. See Figure 13.4-1 for a typical EAA Reservoir A-1 gate/culvert structure schematic. See Section 6.6 for a detailed discussion on the size and hydraulic capacity of the EAA Reservoir A-1 gate structures.

Figure 13.4-1 Typical Gate/Culvert Structure



13.5 *SEEPAGE PUMPING*

The configuration for handling collected seepage is discussed in detail in Section 6.8.

13.6 *GLOSSARY OF TERMS*

The following definitions apply to this specification and generally comply with ANSI/HI 2.1 - 2000 and ANSI/HI 2.6.3 - 2000.

- Best Efficiency Point (BEP): The rate of flow at which the pump efficiency (ηp) is maximum
- Bowl Assembly Efficiency (ηba): The efficiency obtained from the bowl assembly, excluding losses within the pump components:
- ηba = <u>Pw (pump output power) + other pump component mechanical and hydraulic losses</u> x 100 Pba (bowl assembly input power)
- Bowl Assembly Input Power (Pba): The bowl assembly input power is the horsepower delivered to the bowl assembly shaft. See HI Engineering Data Book, second edition for an estimate of the line-shaft horsepower losses.
- Driver Input Power (Pdrv): The electrical power input to the motor driver (kilowatts), or pump input power, (Pp), divided by the speed reducer efficiency.
- Head (h): Head is the expression of the energy content of the water referred to a datum elevation. It is expressed in units of energy per unit weight of the water. The unit used in this specification shall be feet, (ft.) of water.
- Minimum Submergence (S): The minimum water height over the suction bell inlet (Reference ANSI/HI 9.8.7.3 – 1998.)
- Overall Efficiency (ηoa): This is the ratio of the energy imparted to the liquid, (Pw) by the pump to the energy supplied to the driver (Pdrv); that is the ratio of the water horsepower to the power input to the primary driver expressed in percent.
- Preferred Operating Region (POR): The preferred operating region is the region over which the pump's vibration, noise, and cavitation are within acceptable limits.
- Pump Efficiency (ηp): The ratio of the pump output power (Pw) to the pump input power (Pp); that is, the ratio of the water horsepower to the brake horsepower expressed in percent.
- Pump Input Power (Pp): The pump input power (brake horsepower), is the power needed to drive the complete pump rotating assembly including the propeller-bowl assembly input power, line shaft power loss, stuffing box loss and thrust bearing loss. With pumps that rely on the driver thrust bearing, the thrust bearing loss shall be added to the power delivered to the pump shaft.
- Pump Output Power (Pw): The power (water horsepower), imparted to the liquid by the pump.

- Pump Total Discharge Head (hd): The pump total discharge head (hd) is the sum of the discharge gauge head (hgd) measured after the discharge elbow plus the velocity head (hvd) at the point of the gauge attachment plus the elevation head (Zd) from the discharge gauge centerline to the pump datum.
- Pump Total Head (H – commonly called "Total Dynamic Head" or "TDH"): The pump total head, (H) is the difference between the pump total discharge head, (hd) and the total suction head, (hs). This is the measure of the net energy increase per unit weight of the liquid, imparted to the liquid by the pump.
- Rated Condition Point: Rated condition point applies to the rate of flow, pump total head, speed, NPSHR, efficiency, and pump input power as required by the Contract specifications. The rated condition is the point at which the pump manufacturer certifies the pump's performance is within the acceptance criteria tolerances stated in this specification.
- Rate of Flow (Q): The rate of flow, (capacity), of a pump is the total volume through-put per unit of time at the suction conditions. Units used in this specification shall be U.S. gallons per minute, (gpm).
- Shutoff Head: The pump total head (H) when the pump operates at the rated speed and the pump is at zero flow.

  Speed (n): The number of revolutions of the shaft in a given unit of time. Speed is expressed as revolutions per minute, (rpm).
- Total Suction Head (hs), Open Suction: The total suction head (hs) at datum is the vertical distance in feet from free water surface to datum. The average velocity head of the flow in the intake shall be neglected.

SECTION 14

ELECTRICAL DESIGN

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FIGURE

14. ELECTRICAL DESIGN

14.1 DESIGN CRITERIA

14.1.1 Utility Power

FPL overhead 13.2 Kilovolt (kV) power lines are known to exist at the current G-370 pump station which is approximately 5 to 6 miles south of the proposed northeast pump station. Preliminary contact with FPL was made to inform them of the proposed pump station and the anticipated power demands. No additional information was received from FPL at this time concerning what overhead lines would be extended to serve the new pump station.

To stay consistent with existing pump stations, we will request that FPL furnish and install a step-down station transformer at the northeast pump station. The secondary will be requested to be wye connected with the neutral solidly grounded at the transformer installation.

14.1.2 Station Equipment voltage

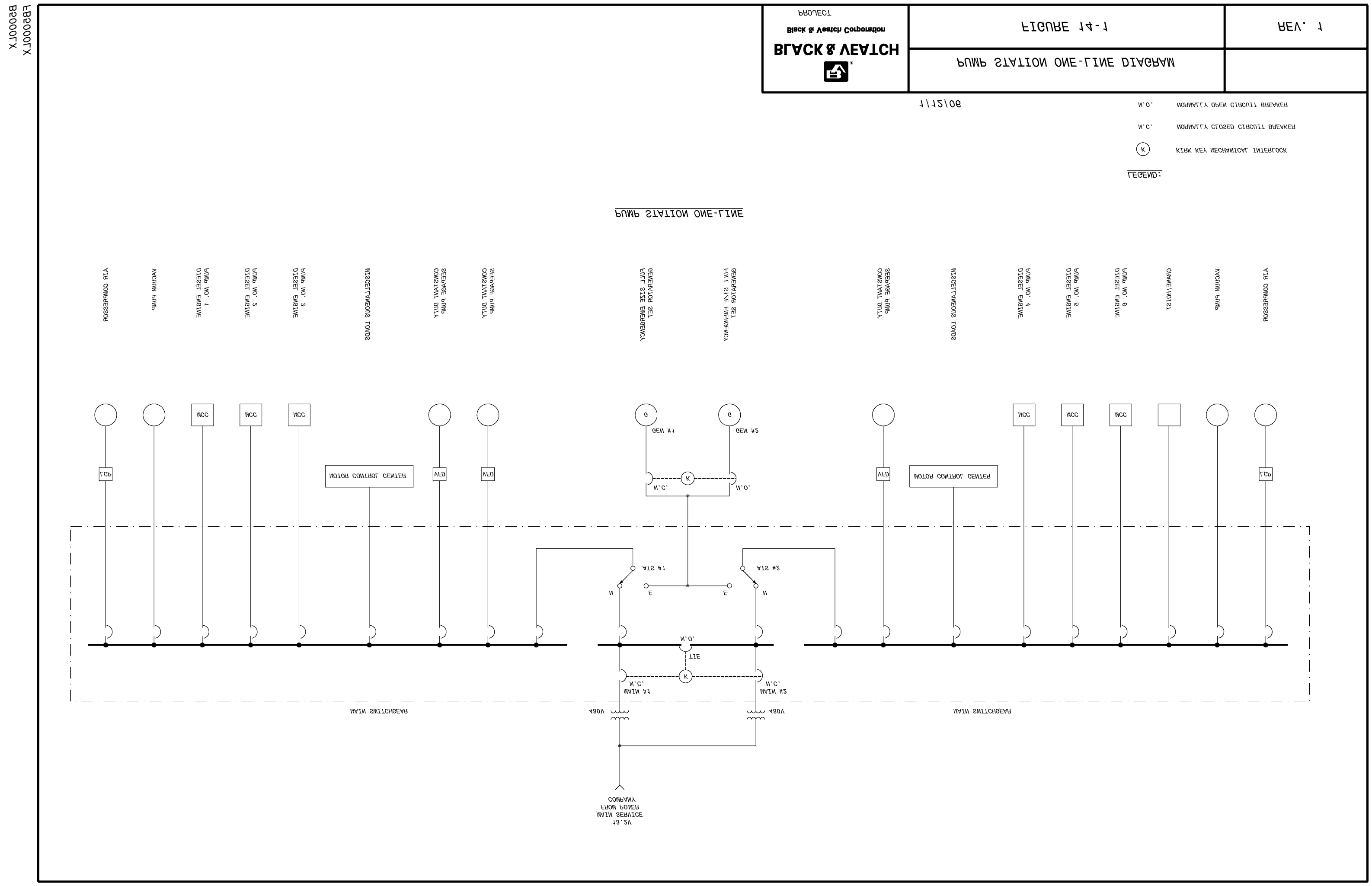
Pump station voltage received from FPL will be 480 volts, three phase, 60 hertz. In general, station equipment voltages will be specified to operate at the following voltages.

Motors rated one Horsepower (Hp) and larger	460 volts, 3 phase
Motors less than one Hp	115 volts, 1 phase
Lighting	115 volts, 1 phase
Convenience receptacles	115 volts, 1 phase

14.1.3 Pump Station Power Distribution

A preliminary pump station one-line diagram (Figure 14.1-1) is included on the following page. The distribution system will be serviced by the FPL transformer serving a split bus. Normally the tie breaker is open and the two main breakers are closed, thus dividing the loads between the two main breakers. To prevent voltage back-feed from the two generators, only two of the three breakers can be closed at the same time. This is mechanically accomplished via key interlocks.

Figure 14.1-1 Preliminary Pump Station One-Line Diagram



14.1.4 Station Switchgear

A switchboard consisting of circuit breakers will be provided to distribute 480 volts of power to various loads, including but not limited to the following equipment:

- Two vacuum system pumps
- Two air compressors
- Six motor control centers for diesel engine pump support equipment.
- Three seepage pumps
- Two motor control centers for miscellaneous loads
- Crane and hoist

The two motor control centers for miscellaneous loads will supply power to individual pumps that are not part of a vendor supplied package and other loads as indicated below. The list of equipment is tentative and subject to change during final design.

- Building supply fans
- Building exhaust fans
- Two waste fuel oil pumps
- Fuel oil receiving pump
- Two lube oil supply pumps
- Lube oil receiving pump
- Four cooling water pumps
- Two fresh water pumps
- Two water lubrication pumps
- Two potable water pumps
- Two lube oil pumps.
- Two waste lube oil pumps
- Generator block heater
- Two traveling trash rakes
- Eight rotating strainers
- Two lighting panels (120/208 volt, three phase)
- 14 motor operated valves
- Water heater

- Instrument air compressor
- Two engine cooling water valves
- Hvac power panel (120/240 volt, single phase)
- Fire alarm and security system power panel (120/240 volt, single phase)
- Seepage bay draw-down pump receptacle
- Drainage pump bay-drawdown pump receptacle
- Seepage pump discharge pump bay receptacle
- Cooling water pump receptacle
- Other loads as required

14.1.5 Standby Generator Power

In addition to normal utility power, the pump station will have two diesel engine powered generators. Each will be sized to operate the station systems should the normal utility power fail. One generator will operate at a time automatically, with the second serving as a stand-by. Fuel storage requirements will be based on a single generator operation for a minimum of seven days.

Each generator will be sized to meet the stand-by requirements of the pump station. This will probably include operation of all plant loads and firm capacity for the seepage pumps.

Upon failure of the utility power, a transfer switch will automatically transfer power supply from a generator. A manual generator start will be provided to exercise the unit.

14.1.5.1 Motor

Motors will be totally enclosed, fan cooled, and of premium efficiency. All outdoor motors will have integral space heaters. Indoor motors five Hp and larger will have integral space heaters.

14.1.5.2 Monitors

The 480 volt switchboard and the motor control centers will each have a power monitor that will provide line and phase voltages, phase currents, kilowatt (kW), kilovolt-ampere reactive (kVAR), power factor, and kilovolt-ampere (KVA).

14.1.5.3 Lighting and Receptacles

Lighting panel boards will be rated for 120/208 volts, three phase. Bus bars will be copper. Circuit breakers will be thermal magnetic bolt-on type.

High bay areas of the pump station will be provided with metal halide light fixtures. The pipe gallery area will have fluorescent light fixtures. Control room, break room, and offices will have T-8 fluorescent light fixtures. Outdoor light fixtures will be wall mounted and controlled by a photoelectric switch. The diesel tank storage area lighting will be pole mounted metal halide fixtures. Lighting levels will be in accordance with the USACE EM 1110-2-3105, Chapter 21 Standard.

Major paths of exit will have LED type exit signs on a dedicated circuit. Emergency lighting will also be provided on this dedicated circuit.

Switches used for lighting will be rated 20 amperes, 120 volts. Duplex receptacles will be rated 20 amperes, 120 volts. Ground fault circuit interrupter (GFCI) type receptacles will be used outdoors and in the restrooms. Office receptacles will have stainless steel plates. Outdoor receptacles will have “in-use” weatherproof covers.

14.1.5.4 Conduits and Wiring

Conduits above grade will be rigid steel. Conduits below grade will be Polyvinyl Chloride (PVC) Schedule 40 pipe. Underground conduits, in general, will be encased in concrete.

Liquid-tight flexible metal conduit will be used at all motors, transformers, and any other equipment that can vibrate or move. Rigid steel conduits will be terminated at equipment and boxes with insulated plastic bushings. The cable tray will be reviewed for use in the pump station during final design.

Wire for 480 volt power applications will be thermoplastic high heat resistant nylon coated (THHN)/thermoplastic heat and water resistant nylon coated (THWN) insulation with stranded copper conductors. The minimum size wire will be 12 gauge.

Wire for control and alarm circuits will be multi-conductor type THHN/THWN insulation, with stranded copper conductors, and a nylon jacket suitable for installation in either a tray or conduit. The minimum size wire will be 14 gauge.

Wire for milliamp (mA) /millivolt (mV) circuits will be single pair shielded instrument cable, type Thermoplastic Fixture Wire Nylon Jacketed (TFN) insulation, with stranded copper conductors, and a nylon jacket suitable for installation in either a tray or conduit. The minimum size wire will be 16 gauge.

14.1.6 Building Systems

14.1.6.1 Lightning Protection

The building will have air terminals on the roof interconnected with copper conductors.

14.1.6.2 Grounding

A ground ring will be installed around the pump station consisting of 4/0 copper cable and ground rods to establish a resistance of five ohms or less. The building’s steel columns, steel rebar in the footing, water piping, lightning protection system, motors, panels, transformers, etc. will be connected to the ground ring in accordance with the National Electric Code.

14.1.6.3 Fire Alarm System

A zoned, supervised fire detection and alarm system will be installed. Ionization type smoke detectors will be used in the pump room and the generator room. To protect against false alarms, the detectors in these rooms will be cross-zoned so that two detectors must be initiated before an alarm is sounded.

14.1.6.4 Closed Circuit Television System

A closed circuit television (CCTV) system, similar to the system at G-370 pump station, will be installed in the northeast pump station.

14.1.6.5 Electrical Design

In the electrical design of the pump station, the motor control center (MCC) and seepage pump's circuit breakers are located on the main switchboard to allow for the two emergency generators to operate all pump station loads, including all three seepage pumps when both generators are operating. If only one generator is running, then all plant loads and two seepage pumps can be operated.

14.1.6.6 Materials of Construction

FPL will provide the service transformer.

The switchboard manufacturer should have a distributor and authorized service representative within the State of Florida. The equipment should be manufactured in the United States. Acceptable manufacturers will be Square D, Siemens, Cutler Hammer, Allen-Bradley, General Electric, or approved equal.

Lighting and 480 volt distribution panel boards and lighting fixtures should be made in the U.S. Light fixtures will be industrial grade.

Generators will be Cummins, Onan, Caterpiller, Detroit Allison, or approved equal. Automatic transfer switches will be Asco, Zenith, or approved equal.

Rigid steel conduit will be Allied, Robroy, Triangle, or approved equal. PVC conduit will be Carlon, Certain-Teed, or approved equal. Liquid-tight conduit will be Electri-Flex, Carol Cable, Anamet, or approved equal.

Wire and cable will be Okonite, Alpha, or approved equal.

14.2 GATE/VALVE OPERATORS AND CONTROLS

The gate and valve operators will be similar to Limitorque operators which have an integral reversing starter, limit switches, control power transformer, open, stop, and close pushbuttons, and local-remote selector switch. The operators will require 480 volt, three phase power from the MCCs. A local mounted safety disconnect switch will be provided near each operator.

14.3 PUMP STATION ENGINEERING GUIDELINES

The SFWMD has in place a standard titled "Major Pumping Station Engineering Guidelines" dated November 29, 2004. That document was used in preparing this basis of design Section.

SECTION 15

INSTRUMENTATION AND CONTROLS

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15. INSTRUMENTATION AND CONTROLS

15.1 DESIGN CRITERIA

This Section defines the instrumentation and controls design criteria for the water control facilities, pump stations and telemetry systems. All systems will be designed in accordance with SFWMD standards. All systems and facilities, as general practice, will be monitored and controlled from a local control system in the pump station. The local control system will be Programmable Logic Controller (PLC) based. Monitoring and control will be available from the Remote Terminal Unit (RTU) of the SFWMD Supervisory Control and Data Acquisition (SCADA) facilities. The existing telemetry system centralized at the SFWMD headquarters will be extended to include the new facilities. Instrumentation and control features will include the following features:

- The master control PLC will be an Allen-Bradley ControlLogix 1756 PLC system. Packaged systems in the pump station will be provided with stand-alone Allen-Bradley SLC-5/04 (or possibly SLC-5/05 with Ethernet) controllers
- Monitoring and control of remote sites, including gated spillways, gated culverts, and monitoring stations will be over Data-linc spread spectrum radios. Equipment will be controlled by the Master PLC, and passed through to the RTU
- Pumps will have control from the SFWMD Control Center, through the RTU. Gated structures will also have control through the Control Center
- Analog control signals will be 24 VDC, 4-20 mA. Discrete signals will be 24 volt, direct current (VDC). Interposing relays shall be used where necessary to provide isolation and conversion to 24 VDC. Discrete output signals will interface field devices through interposing relays. Surge suppression shall be provided for all instrumentation. The SFWMD design details will be followed

15.2 RESERVOIRS AND CANALS

The level of the EAA Reservoir A-1 and all canals associated with the pumping stations will be monitored through a Campbell Scientific RTU. The signal will be transmitted to the SCADA system for display.

The SFWMD standard for level monitoring is the Balluff linear potentiometer, located in a stilling well. For variable applications exceeding 20 feet, the Rittmeyer pressure transmitter provides increased range. Stilling wells shall be installed in accordance with the SFWMD design details. Water levels in the embankment will be monitored with piezometers. Water quality monitoring will be provided as outlined in the SFWMD design details.

15.3 WATER CONTROL FACILITIES

Monitoring and control of gated spillways will be through the RTU. Control of the gates will be manual from the control system, and will be based on the water level in the EAA Reservoir A-1.

Monitoring and control of gated culverts will be through Campbell Scientific RTUs.

15.4 PUMP STATIONS

Pump station control and monitoring can vary from the simple manual operation of an agricultural style station to the more complicated automatic or remote operation of a typical SFWMD station. There are also varying degrees of complications for the remote operation of a station. Electric motor drivers have far less auxiliary systems than a diesel engine driven pump station and therefore, a much simpler control and monitoring system. Electric driven pump stations are also typically not used for flood control applications due to the possibility of power outages during a storm event. Therefore, pump system reliability of a flood control station does not apply to an electric motor driven station. SFWMD pump station auxiliary systems and the start/stop of the driver are controlled by a PLC. The receiving and sending of the control and monitoring data are via an RTU.

15.4.1 Operation

In the more recently constructed SFWMD flood control stations, there are typically three modes of operation: local/auto, local/manual and remote. Local/auto operation consists of operating the engine with the auxiliary systems automatically controlled from a remote facility. Local/manual operation consists of operating each component of the pumping system by using manual controls. Remote operation consists of operating the engine via the SFWMD’s telemetry system with the auxiliaries automatically controlled. Remote or manual start-up and shutdown, as well as alarm shutdown sequences for the engine are automatically controlled by the engine’s electronics.

Emergency shutdown of the engine requires an immediate deceleration of the engine speed to zero. The control panel includes a hard-wired emergency shutdown system to shutdown the engine in the event that a failure occurs and the engine controller fails to initiate its own shutdown sequence. The emergency shutdown also functions when the “manual-auto-off” switch is in the manual mode. The circuit generates a shutdown on the actuation of the emergency stop pushbutton. The trash rake and fuel auxiliary systems are also shutdown during an emergency stop condition.

15.4.2 Diesel Engine Driver Control and Monitoring System

The diesel engine’s electronic control module (ECM) provides monitoring of vital engine parameters and control of engine operation. The ECM output is data-linked to the engine PLC through a converter that translates automotive protocol standard SAE-J1939 serial data to the required digital input of the input/output (I/O) modules of the PLC. The ECM J1939 output data includes all monitoring data, diagnostic information, and operating history and can be displayed by the PLC’s monitor. The engine PLC through a converter shall be networked to the RTU (MOSCAD) for communication to the SFWMD’s operational center. The RTU (MOSCAD) typically has a monitor to display all station data. The various auxiliary functions, such as low water level shut-down, trash rake operation and alarm, and high reduction gear oil temperature or vibration, are monitored and controlled by the engine’s PLC. The PLC shall have the capability to automatically shut the engine down given an alarm condition. Instrumentation signals for station monitors such as stage data, electrical service power phase monitoring, and fuel storage tank levels shall be connected directly to the RTU (MOSCAD) unit.

15.4.2.1 Logic Controller (PLC)

The microprocessor based controller shall be an Allen-Bradley model or equal for machine level control applications requiring limited I/O quantities and limited communications capabilities. The PLC shall be mounted in the engine control panel and shall provide control and monitoring of the engine, transmission, and the auxiliary systems. The engine ECM I/O serial data via a converter (J1939/DF1-RS232) shall be linked to the I/O module of the controller. The PLC is linked to a DH485 network and with the RTU (MOSCAD) via a converter (DH485/DF1). A station PLC shall be located in the control room with control and monitoring capabilities for all station systems.

15.4.2.2 Display Panel

An Allen-Bradley display panel model or equal will be provided for each PLC. The display will be mounted in the engine control panel door.

15.4.2.3 Converter (J1939/DF1-RS232)

A communication device will be provided to convert the J1939 serial communication data to the DF1-RS232 digital input that is required by the PLC.

15.4.2.4 Interface Module (DH485/DF1)

An Allen-Bradley 1770-KF3 module will be provided for conversion of the network DH485 digital output to the serial DF1 communication signal that is required by the RTU (MOSCAD).

15.4.2.5 Surge Suppressor

A surge suppressor will be provided that will protect the I/O modules of the PLC from lightning induced surges, electrical fast transients and EMI/RFI noise. The surge suppressor shall meet or exceed highest class severity level of IEC 1000-4-4 and 1000-4-5. The suppressor shall be UL-497B listed. The surge protector will be Circuit Components, Inc.'s "Surge Control SAB Series" or SFWMD approved equal.

15.4.2.6 Vibration Switch

Each reduction gear shall be provided with a vibration control switch to protect the equipment from damaging shock or vibration. Each switch shall be a 24 VDC powered electro-mechanical device, with two Single Pole, Double Throw (SPDT) snap acting switches rated at 2A up to 30 VDC, and mounted in a National Electrical Manufacturers Association (NEMA) four enclosure. Each switch shall have a remote reset to allow reset of the tripped unit from a remote location, an adjustable time delay to override trip operation for a preset length of time (to prevent trips during transient pump cavitation events, for instance), and a fine adjustment to precisely select the degree of sensitivity.

15.4.2.7 Temperature Monitors

The temperature probes provided shall be resistance temperature detectors (RTDs) and shall comply with ANSI 34. RTDs shall be 100 ohm 3-wire platinum in a Type 304 stainless steel sheath with watertight connection head.

15.4.2.8 Liquid Level Gage

A combination liquid level gage with adjustable low limit switch will be used to provide visual indication of oil level and signal low level conditions.

15.4.2.9 Indicating Level Switches

Indicating level switches shall be provided to signal a low level condition. Water level sensors shall be installed to signal a low level condition. Sensors shall be floatless, pressure sensitive, diaphragm actuated switches.

15.4.3 Monitoring Instrumentation

Compliance monitoring locations shall be provided with auto samplers. The auto sampler will be activated by the level at the monitoring location. Auto samplers will include all necessary facilities for access, including platforms and stairs.

15.4.4 Station Emergency Power

Given a power outage, the flood control station requires emergency electric power backup. Typically an adequately sized engine generator is provided for backup station service during the time that utility power is lost. In a large flood control station, redundant system is provided. Controls are provided in the 125 ampere Automatic Transfer Switch (ATS) to either manually or automatically start the engine-generator. A RTU output command will remotely STOP the engine generator should the unit unnecessarily start.

Electric motor driven seepage pump stations intended for flood control shall utilize a backup generator to operate the pump station during a loss of utility power.

15.4.5 Stage Monitors

Upstream and downstream stilling wells with water level transmitters provide analog water level data of the approach and discharge channels. A stilling well with a water level transmitter is also provided in each pump intake to monitor low water levels. The water level transmitters provide a proportional 4-20 mA signal. The stilling well water level transmitters are normally continuously powered from a 120 volts, alternating current (VAC) to 24 VDC power supply. Upon the loss of 120 VAC service, the RTU's "PULSE ANALOG CIRCUITS" program will intermittently power and scan the analog water level transmitters.

15.5 TELEMETRY

The SCADA system RTU will be located in the building close to the antenna. The RTU will be a Motorola MOSCAD or equal unit to be compatible with the existing units already installed at other SFWMD locations. The pump stations will be remotely monitored through the SFWMD's SCADA system. This is the SFWMD's proprietary system consisting of an RTU and an antenna. The RTU will be capable of transmitting data to a main station via radio. Data to be transmitted is to be determined. SFWMD may require the remote control of the station and the SCADA system, of the station should provide for this type.

SECTION 16

ARCHITECTURAL

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16. ARCHITECTURAL

16.1 DESIGN CRITERIA

16.1.1 Introduction

The northeast pump station building will be constructed to accommodate the pumps, motors, generators, and ancillary systems. In addition, adequate area will be provided for a control room, offices, break room, toilet, locker/shower and mechanical equipment.

16.1.2 Design Requirements

16.1.2.1 Codes and Standards

Design and specifications of all work will be in accordance with the latest laws and regulations of the federal government, applicable state and local codes and ordinances, and applicable industry standards. Other recommended standards will be used where required to serve as guidelines for design, fabrication, and construction when not in conflict with the above standards. The building will be designed in accordance with Florida Accessibility Code and Americans with Disabilities Act Accessibility Guidelines (ADAAG). Anti-Terrorism/Force Protection measures for the building will be addressed during the 30 percent design phase. There is no requirement to incorporate the principles of Sustainable Design and Development for the building.

16.1.3 Life Safety

The building will be designed to meet the minimum construction and life safety requirements as required by the applicable codes and criteria. Appropriate type, size, and quantity of fire extinguishers will be provided in compliance with all applicable fire and life safety codes; including a sprinkler system in designated areas. For details, refer to Section 17.

16.1.4 Material and Life Cycle

The building shall be designed to minimize life cycle cost, energy consumption, and maintenance through proper selection of mass, form, materials, and construction standards. Integrally colored materials shall be used as much as possible to eliminate painting. The design life of the building shall be a minimum of 50 years. Refer to Sections 11.3.5 and 11.3.6 for seismic and wind loading design criteria. The service life span will be the same as the building service life, except for the following: protective elements, wall primary weather-barrier elements, joint sealers, surfaces exposed to view, and roof covering weather barriers. These will have varying service lives, as shown in Table 16.1-1.

Figure 16.1-1 Exceptions to the Building Service Life Span

Material	Life Cycle
Protective Elements	Minimum 20 Years
Wall Primary Weather-Barrier Elements	Minimum 50 Years Functional And Aesthetic Service Life, Excluding Joint Sealers
Joint Sealers (fuel resistant)	Minimum 20 Years Before Replacement
Surfaces Exposed to View	Minimum 20 Years Aesthetic Service Life - No Color Fading, Crazing and Delamination of Applied Coatings
Roof Covering Weather-Barriers	Minimum 20 Years, Fully Functional

16.2 EXTERIOR ARCHITECTURAL FEATURES

16.2.1 Shell

The elements forming usable enclosed space and separating that space from the external environment comprise the shell and consist of:

16.2.1.1 Superstructure

The superstructure includes all elements forming floors and roofs above grade, and the elements required for their support, insulation, fireproofing and fire stopping. The structural system for the superstructure shall be a steel frame with reinforced CMU walls and poured in place concrete roof and shall be designed in accordance with the applicable building codes as defined in Section 11.5.

16.2.1.2 Exterior Enclosure

The exterior enclosure includes all essentially vertical elements forming the separation between exterior and interior conditioned space, including exterior skin, components supporting weather barriers, and jointing and interfacing components; not including the interior skin unless an integral part of the enclosure. The exterior enclosure will be fully grouted reinforced CMU wall with an application of latex block filler and an acrylic coating on all exposed surfaces. Thermal performance for the exterior enclosure is not applicable to main equipment rooms. Exterior enclosures will be insulated for all air-conditioned spaces.

All exterior doors will be painted, hollow metal doors with painted metal frames. Insulated doors will secure air-conditioned spaces. Overhead doors shall be roll-formed galvanized steel construction, electrically operated and shall be sized to fit the largest equipment for the building. Louvers will be designed as required for ventilation of the spaces and equipment. The building wall openings for fans and louvers will have missile barrier protection over screens constructed to withstand 155 mph wind loading and windborne debris in accordance with the wind load design criteria specified in Section 11.3.6. All doors and louvers will be hurricane impact resistant.

16.2.1.3 Roofing

Roofing includes all elements forming weather and thermal barriers at horizontal roofs, decks, and roof fixtures. A single ply roofing membrane will be used over the reinforced poured concrete roof deck. The roof will be sloped to stainless steel drain scuppers formed through the parapet. The roof runoff is directed down the walls via downspouts made from hollow structural tubing to resist missile impact during hurricane events. All flashing, trim, and accessories will be of stainless steel sheet metal. Access to the roof will be provided by a roof hatch and will be controllable by authorized personnel only.

16.3 INTERIOR ARCHITECTURAL FEATURES

16.3.1 Floor

All floor slabs will be sealed reinforced poured concrete.

16.3.2 Partitions

Partitions provided for physical separation between spaces will be constructed to achieve fire ratings required by code; appropriate security between adjacent spaces; and visual, acoustical, olfactory, and atmospheric isolation as necessary to maintain desirable conditions in each space. Partitions will comprise the following elements: Fixed partitions of fully-grouted, reinforced, full-height CMU; and partial height partitions of fixed, solid, opaque visual barriers for toilet compartments. The control room will have glass panels to allow the operator an unobstructed view of the operation floor. The control room/break room will be designed for sound proofing with a minimum Sound Transmission Coefficient (STC) of 49.

16.3.3 Interior Doors and Windows

All interior doors shall be painted, hollow metal doors with painted metal frames. Interior windows will be provided between adjacent spaces. Fixed interior windows and operable interior windows, when closed, will function as partition elements and will not degrade performance of partitions below the levels specified. Sound insulated doors and windows will be provided to meet the STC of not less than 49.

16.3.4 Interior Finishes

16.3.4.1 Offices/Control Room/Break Room

- Wall: Painted
- Floor: Non-skid ceramic tiles
- Ceiling: Suspended acoustical ceiling tiles

16.3.4.2 Toilets/Showers

- Wall: Ceramic tiles
- Floor: Non-skid ceramic tiles
- Ceiling: Moisture resistant gypsum board

16.3.4.3 Locker Room

- Wall: Painted
- Floor: Non-skid ceramic tiles
- Ceiling: Moisture resistant gypsum board

16.3.4.4 Equipment Room/Maintenance Shop/Janitor's Closet

- Wall: Painted
- Floor: Sealed concrete
- Ceiling: None. All exposed concrete will be painted

16.3.4.5 Fan/Filter Rooms

- Wall: Painted
- Floor: Sealed concrete
- Ceiling: None. All exposed concrete will be painted

16.3.4.6 Vertical Circulation

Stairs will be provided for access to mechanical spaces and equipment mezzanines. Also, a vertical lift that meets accessibility requirements will provide access to the control room.

16.4 INTERIOR FIXTURES

Interior fixtures permanently attached to interior walls, ceilings, and floors, except for equipment items, will be provided and comprise the following elements:

16.4.1 Identifying Devices

Informational accessories, including room numbers, signage, and directories.

16.4.2 Storage Fixtures

Items intended primarily for storing or securing objects, materials, and supplies, including cabinets, casework, and shelving.

16.4.3 Accessory Fixtures

Specialty items intended to provide service or amenity to building interiors, including toilet and bath accessories, visual display surfaces, and telecommunications fixtures.

16.4.4 Interior Fixtures

Other items fixed to the interior construction that enhance comfort or amenity in building spaces.

SECTION 17

HVAC, PLUMBING, AND FIRE SUPPRESSION SYSTEMS

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17. HVAC, PLUMBING AND FIRE SUPPRESSION SYSTEMS

17.1 DESIGN CRITERIA

The following describes the basis of mechanical design and criteria associated with the heating, ventilating, and air conditioning (HVAC); plumbing; and fire suppression systems for the new northeast pump station. Table 17.1-1 details the EAA Reservoir A-1 Project site design criteria; Table 17.2-1 details the indoor design criteria for the EAA Reservoir A-1 Project.

Table 17.1-1 EAA Reservoir A-1 Project Site Design Criteria

Site Elevation	
Above sea level, feet NAVD88	16 to 27
Site Location	
North latitude, degrees	26
West longitude, degrees	81
Ambient Design Temperatures (1)	
Winter, design dry bulb, degrees F	42
Summer, design dry bulb/mean coincident wet bulb, degrees F	93/77
Dehumidification, design dew point, degrees F	78
Degree Days	
Heating (Base 65 degrees F), days	418
Cooling (Base 50 degrees F), days	8,924
Rainfall Intensity (2)	
Actual, inches/hour	4.7
Design, inches/hour	5

(1) The winter and summer design temperatures are based on the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) frequency levels 99.6 percent and 1 percent, respectively.

(2) The actual rainfall intensity rate is based on a 60 minute duration and 100 year return period.

17.2 HVAC

The following is a description of the HVAC systems:

17.2.1 Heating Systems

Electric wall heaters will be provided in the Men's and Women's Toilet/Locker rooms for supplemental heat.

17.2.2 Ventilation Systems

A forced air ventilation system will be provided for the operating floor area of the pump station. The system will utilize centrifugal fans for supply and propeller fans for exhaust. The supply air system will consist of louvers for air intake, automatic roll filters for filtering, centrifugal fans for supply, and a below floor air distribution header for supplying air to the operating floor area.

The roll filters and supply fans will be located in rooms along the wall opposite from the pump's engines. The exhaust fans will be located high above the floor on the engine side of the pump station. The ventilation system will remove the heat gains from the equipment as well as supply make up air for the engine air intakes.

The intake and exhaust louvers will be Miami-Dade County approved, and will be provided with missile barriers.

The ventilation system fans will be controlled by their individual ON-OFF-AUTO selectors switches. When the exhaust fan selector switches are in the "AUTO" position, the exhaust fans will be interlocked with the controls for the supply fans. When the supply fan selector switches are in the "AUTO" position, the quantity of supply fans operated will be automatically controlled based upon the quantity of engines operating in the engine pump room and controlled by the room thermostats in the engine pump room.

17.2.3 Air Conditioning Systems

The air conditioning systems will be split system type heat pumps. A heat pump will be provided for the shop, control room, break room, locker room, and restroom. The heat pump serving the break room, locker room, and restroom will also be ducted to provide a backup to the control room's air conditioning system. Each heat pump will be provided with a backup emergency electric heating coil. Each unit will be controlled by a remote wall mounted thermostat to maintain the desired space temperature.

The air handling units and heat pumps will be located inside the pump station.

The locker room, restroom, and janitor's closet will be exhausted by duct fans ducted to exhaust louver or wall caps.

Table 17.2-1 Indoor Design Criteria for EAA Reservoir A-1 Project

Area	Design Temperatures (F) [1]			Ventilation Requirements	Ventilation Notes
	Summer	Winter			
	Design	Design	Setpoint		
Engine Pump Room	100	50	50	1.5 cfm/sf (C)	Note 2
Shop	78	72	72	--	Note 4
Janitor's Closet	100	--	--	--	Note 3
Control Room	70	70	70	--	Note 4
Break Room	78	72	72	--	Note 4
Locker Room	78	72	72	--	Note 3
Restroom(s)	78	72	72	--	Note 3

AC/HR = designates air changes per hour
(C) = designates the ventilation system operates continuously
(I) = designates the ventilation system operates intermittently

[1] Indoor conditions reflect operating temperatures for personnel comfort, code/standard recommendations, or equipment protection.

Notes:

1. The ventilation system will be sized on the more restrictive of the AC/HR (or cubic feet per minute per square foot – cfm/sf) listed, or the airflow required to maintain the indoor design temperature based on the summer outside design temperature
2. Additional intermittent ventilation will be provided as required to maintain the indoor design temperature based on the summer outside design temperature, or to meet the engine combustion air requirements
3. The exhaust rate will be based on the most stringent requirement of: 0.5 cfm/sf of floor area; 50 cfm per toilet or urinal; or 100 cfm minimum
4. The ventilation rate will be based on the exhaust requirements or as required by ASHRAE 62-1989, whichever is more stringent

17.3 POTABLE WATER

Investigation of potable water usage at the existing major pumping stations (G-310 and G-370) indicates low demand and infrequent use of potable water. Potable water is supplied to a kitchen sink, restrooms and showers. Bottled water is used for drinking. It was reported that the current potable water systems are sized for more demand than the system experiences, and as a result, the treatment systems are experiencing problems due to a lack of flow.

As an alternative to the potable water supply system installed at the existing pumping stations, which is canal water processed through sand filters and reverse osmosis (RO) membranes, the use of a shallow water well will be reviewed. Treatment of this water could be with aeration, canister filtration, chlorination, and softening. The design will incorporate storage that will serve the typical low demand but also accommodate the infrequent periods of larger demands when the pump station houses personnel during extreme weather events. Changing the potable water source to a well would require water quality sampling and analyzing, and based on the results of the analysis, an appropriate water purification system would be compared to the current RO treatment system. Alternative systems will be considered as part of the 30 percent design.

The potable water system selected shall supply potable water to restrooms, sinks and showers. An electric-powered domestic water heater will be provided to supply water at 120 degrees F to the sinks and showers.

17.4 FRESH WATER SUPPLY SYSTEM

A fresh water system will be provided to supply water for lubricating water, seal water, and hose bibs for area washdown. The fresh water system will be supplied by water from the adjacent canal, and treated using in-line strainers.

17.5 COOLING WATER SYSTEM

Cooling water for use in the pump engines and gear reducers will be provided by strained water from the adjacent canal. This will be a once-through cooling system, and the used water will be collected and discharged back into the canal.

17.6 SANITARY SYSTEM

All plumbing fixtures that require drainage will discharge to the sanitary system. In addition, floor drains located in the locker room and restrooms will discharge to the sanitary system. Floor drains will not be provided in the pump room so that potential oily waste will not be discharged to the sanitary system. Sanitary drainage from the building will be collected in a septic tank. Soil tests will be conducted to verify the efficiency of a leach field. If the soil conditions are not favorable for a leach field, or the amount of discharge is determined to be minimal, the septic tank could be used for storage of wastewater and pumped regularly for removal off site.

17.7 STORMWATER SYSTEM

Storm drainage will be collected from the roof drains and leaders. All storm drainage at the pump station will be routed to the forebays.

17.8 FIRE SUPPRESSION SYSTEM

It is expected that an automatic fire sprinkler and detection system will be required for the entire pump station facility. Further code investigation will confirm this requirement during detailed design. If a sprinkler system is required, a pre-action system will be provided. The sprinkler system shall be installed in accordance with NFPA 13 standards. Portable fire extinguishers will be installed in accordance with paragraph 16.1.3 of this document.

The SFWMD shall review all design assumptions, criteria, and calculations. Verification with the SFWMD and the SFWMD’s insurance underwriter shall be done for the fire protection systems.

SECTION 18

ACCESS AND SECURITY

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18. ACCESS AND SECURITY

18.1 ACCESS

The details of access to and within the EAA Reservoir A-1 have been discussed earlier in Section 12. Public access to the EAA Reservoir A-1 and associated facilities will be for recreational opportunities along U.S. 27, as discussed in Section 19. Public access to the top of the EAA Reservoir A-1 embankment will be provided along the eastern embankment adjacent to U.S. 27 for recreational activities.

The public and SFWMD staff will be able to access the EAA Reservoir A-1 in the following general areas:

- Top of the embankment
- Inside the EAA Reservoir A-1
- Outside the EAA Reservoir A-1between the embankment and the seepage canal

Entrance to these general areas will be provided in the following locations:

- Near the northeast corner of the EAA Reservoir A-1 at the existing road intersecting U.S. 27
- At several locations along U.S. 27
- Near the existing G-370 pump station

Access to the new northeast pumping station will either be provided by the existing intersection or a new intersection with U.S. 27.

In addition, allowances for SFWMD vehicle and personnel access will be required. Access to outlet structures located along the south side of the EAA Reservoir A-1 will be provided by way of a road located along the top of the embankment.

18.2 SECURITY

The EAA Reservoir A-1 and all elements will follow the guidelines of the SFWMD and the U.S. Department of Homeland Security.

18.2.1 Fences and Gates

The EAA Reservoir A-1 is located in a remote rural area and is currently surrounded by undeveloped agriculture land. Pedestrian entry to the EAA Reservoir A-1 embankments on the north and south sides will be discouraged through the use of signs. Vehicle entry to the embankments on the north and south sides will be discouraged through the use of gates. It is not anticipated that the entire EAA Reservoir A-1 will be surrounded by a perimeter fence. The new northeast pumping station and any outlet structures will have controlled access through the use of fences and gates. Fences at the water control structures will be provided with locked gates keyed to match the SFWMD's current lock system. Electric gates and locks are not anticipated. Gates will be provided at vehicle access points located along the EAA Reservoir A-1's embankment.

18.2.2 Site Monitoring, Northeast Pump Station

A closed circuit television system will be employed for security. Cameras will be located at each entrance to the building as well as strategically located within the building. Cameras will also provide views of vehicle entrance gates.

18.2.3 Building Access, Northeast Pump Station

Items that will be considered when controlling access to the building will include:

- Door position switches
- Interior motion sensors
- Keypad access with timed alarm override

All security features and elements will be coordinated with the SFWMD prior to final design.

SECTION 19

PUBLIC INVOLVEMENT AND OTHER CONSIDERATIONS

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19.1 PUBLIC INVOLVEMENT

The EAA Reservoir A-1 Project will have impacts on the surrounding area and local communities. However, implementation of the EAA Reservoir A-1 Project will meet objectives consistent with the ongoing work by the U.S. Army Corps of Engineers (USACE).

The list of stakeholders will continue to be developed as the EAA Reservoir A-1 Project progresses. Stakeholders currently identified include the following:

- Audubon Society of Florida
- Bergeron
- Carroll, Jack E. and Larry G.
- EAA Environmental Protection District (EPD)
- Florida Crystals Corporation
- Florida Department of Agriculture and Consumer Services (FDACS)
- Florida Department of Environmental Protection (FDEP)
- Florida Department of Transportation (FDOT)
- Florida Fish & Wildlife Conservation Commission (FWC)
- Florida Ranch Enterprises, Inc.
- Florida Power & Light (FPL)
- Okeelanta Corporation
- New Hope South, Inc.
- NLDS Acquisition Corporation
- Palm Beach County
- Star Ranch Enterprises, Inc.
- Sugar Farms Co-op
- Talisman Sugar Corporation
- Tri-Cities (South Bay, Pahokee, Belle Glade)
- U.S. Army Corps of Engineers (USACE)
- U.S. Environmental Protection Agency (USEPA)
- U.S. Fish & Wildlife Service (USFWS)
- U.S. Natural Resources Conservation Service (NRCS)
- U.S. Sugar Corporation
- Woerner South, Inc

The NEPA requires full public participation in the planning and decision making process. The USACE established a Public Involvement Program which included the following initiatives for obtaining public input:

i. Establishment of a website (http://www.evergladesplan.org) to provide information and communication paths
ii. Submission of scoping letter to identified EAA Reservoir A-1 Project stakeholders providing a description of the project and identifying points of contact for more information or registering concerns
iii. Two public workshops were conducted in Spanish and English at Belle Glade, Florida in August 2001 and January 2003
iv. A series of Project Team meetings that were open to the public were held within 50 miles of the Project Area
v. As part of the Restudy reconnaissance and feasibility phases of the study, extensive public programs were designed to determine the public's concerns throughout the study process

As this Project progresses, it is anticipated that previous public involvement work, such as that undertaken by the USACE will continue and be expanded upon to address the specific issues of the EAA Reservoir A-1 Project. As of this writing, public stakeholder meetings have been held with:

- FDOT
- FPL
- Palm Beach County (about master plan exemptions)

Stakeholder meetings have been scheduled with environmental and agricultural interests. The SFWMD has also scheduled a Water Resources Advisory Commission (WRAC) issues workshop to discuss the concepts presented in this BODR.

Additionally, an assurance pre-application meeting was held with FDACS. A Criteria Committee Meeting (CCM) and other meetings have been held with representatives from the following agencies present:

- FDEP
- FWC
- USACE
- USEPA
- USFWS

19.2 OTHER CONSIDERATIONS

This Section assesses the options for secondary benefits of the EAA Reservoir A-1 Project. The proposed EAA Reservoir A-1 Project should be designed to minimize negative impacts on existing public use and recreation opportunities where practicable. Enhancement of ecological values is a desirable feature of the EAA Reservoir A-1 Project that is supported and promoted by the USFWS. Opportunities to provide benefits for fish and wildlife resources can be incorporated into the EAA Reservoir A-1 Project and seepage canal design in a number of ways that are described in Technical Memorandum 19-1, and summarized here.

Two major secondary benefits of the EAA Reservoir A-1 Project are identified: 1) habitat creation and enhancement and 2) creation of recreational opportunities.

19.2.1 Habitat Creation and Enhancement

The construction of the EAA Reservoir A-1 will result in aquatic habitat on the 97 acres of shrub and brushland, 206 acres of wetlands, and 30,609 acres of uplands that are currently in agricultural use. The uplands and most of the wetlands on the proposed EAA Reservoir A-1 Project site are providing habitat of limited value to wildlife due to the impacts of extensive agricultural activity. Aquatic habitat quality will be dependent on the quality of water, quantity of water, velocity of water, and physical ground cover. Features of the EAA Reservoir A-1 and canals can be provided to benefit habitat:

- Existing farm canals and the interior borrow canals will maintain minimum levels of water to act as refugia for aquatic organisms and to ensure sustainability of the aquatic habitat
- Construction of seepage canals would aid in establishing desirable vegetative cover, improved water quality and creation of additional substrates
- Littoral benches will be placed intermittently along the seepage canals

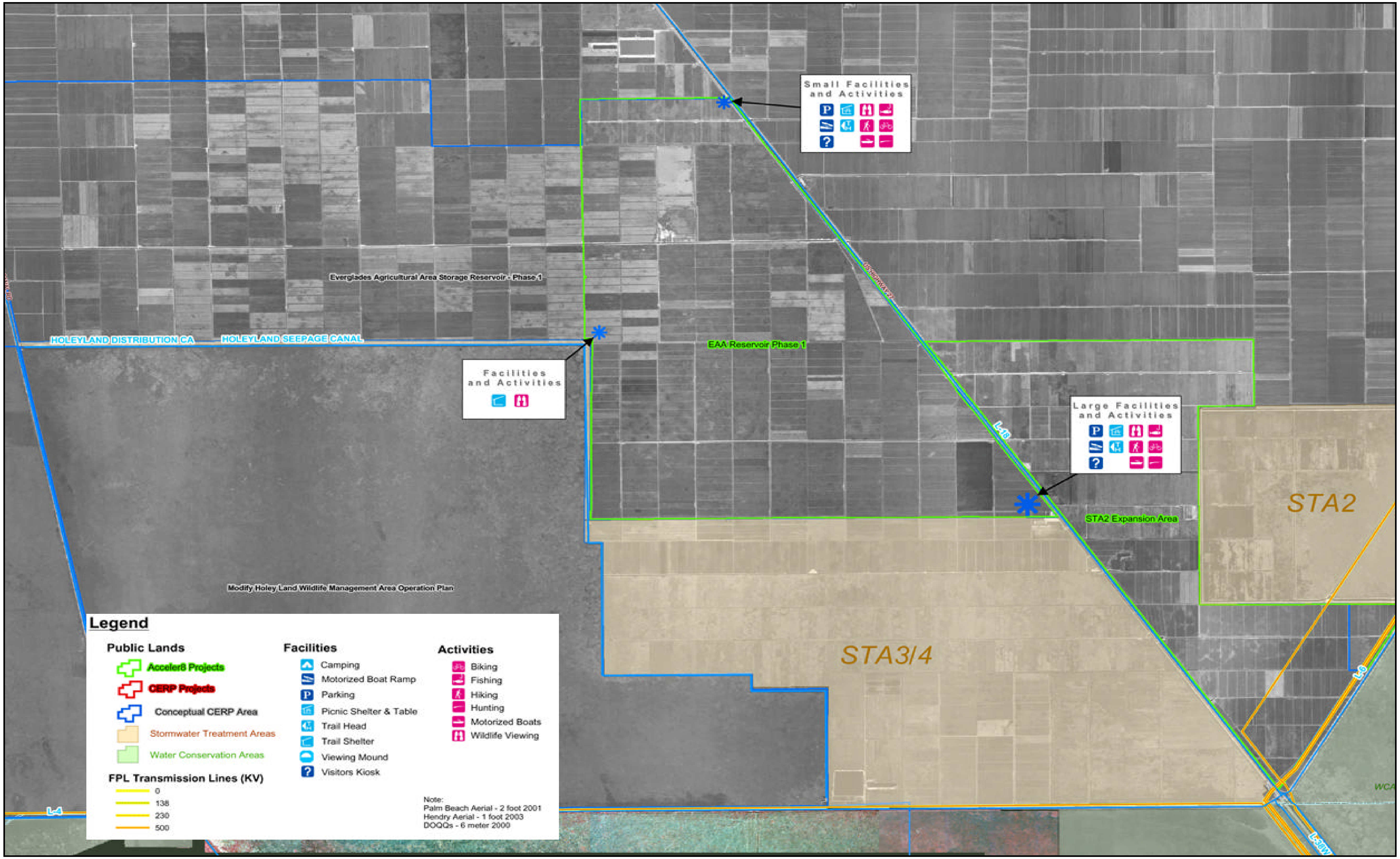
19.2.2 Recreational Opportunities

The proposed project has the potential to provide recreational resources within South Florida; however, recreational uses need to be consistent with the Project goals and objectives. Potential recreational opportunities at the EAA Reservoir A-1 Project site include bird watching, nature trails, interpretative trails, hunting, canoeing/kayaking, hiking, fishing, boating and hunting. See Figure 19.2-1 for the location of potential recreation opportunities.

Fluctuating water levels with in the EAA Reservoir A-1 may greatly influence the potential for boating. Boats will be allowed subject to conditions on boat types and boat operation in order to minimize the risks to water quality and habitat. A decision on design, expected amount of use, and specifically where to locate boat ramps will be made during subsequent design phases.

Replacing the low-value habitat of the existing agricultural areas with native vegetation and open water habitat provided by the proposed EAA Reservoir A-1 and buffer areas will provide significant benefit and recreational opportunities. During the first year of operation, SFWMD staff will evaluate potential recreational opportunities and determine those that are appropriate for the EAA Reservoir A-1 and embankment. These nature-based recreation opportunities will benefit the local economies as dollars are spent on equipment, licenses, and travel.

Figure 19.2-1 Potential Recreation Opportunities



SECTION 20

DRAFT OPERATING PLAN

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20. DRAFT OPERATING PLAN

20.1 PROPOSED FEATURES

The Project Operating Manual (POM) is for day-to-day water management under essentially all foreseeable conditions affecting the EAA Reservoir A-1 Project. The POM for STA-3/4 is a separate document and is not updated for this BODR. The draft POM is developed as part of the BODR for use during the PIR/EIS phase of the EAA Reservoir A-1 Project. Modifications and revisions to the document will occur during the detailed design and subsequent phases. See Figure 20.1-1 for the Phase I EAA Reservoir A-1 location.

Preliminary layout of the EAA Reservoir A-1 includes set backs from the property line to the exterior of the embankment of 425 feet on the north and northern west sides, and 275 feet on the east. The exterior of the south embankment and the southern west sides will start on the top of the northern and eastern Inflow/Supply Canal levee. Therefore, no set back is included.

The EAA Reservoir A-1 is designed for a normal maximum operating depth of 12 feet and total storage of approximately 190,000 acre-feet. The perimeter embankment is approximately 114,000 feet long and is 14 feet wide at the crest, with 3H:1V side slope on each side. Total embankment height is 26 feet above OG to provide for the PMP, wind setup, and wave run-up. Design data for the EAA Reservoir A-1 Project is included in Table 20.1-1.

Table 20.1-1 EAA Reservoir A-1 Project Design Data

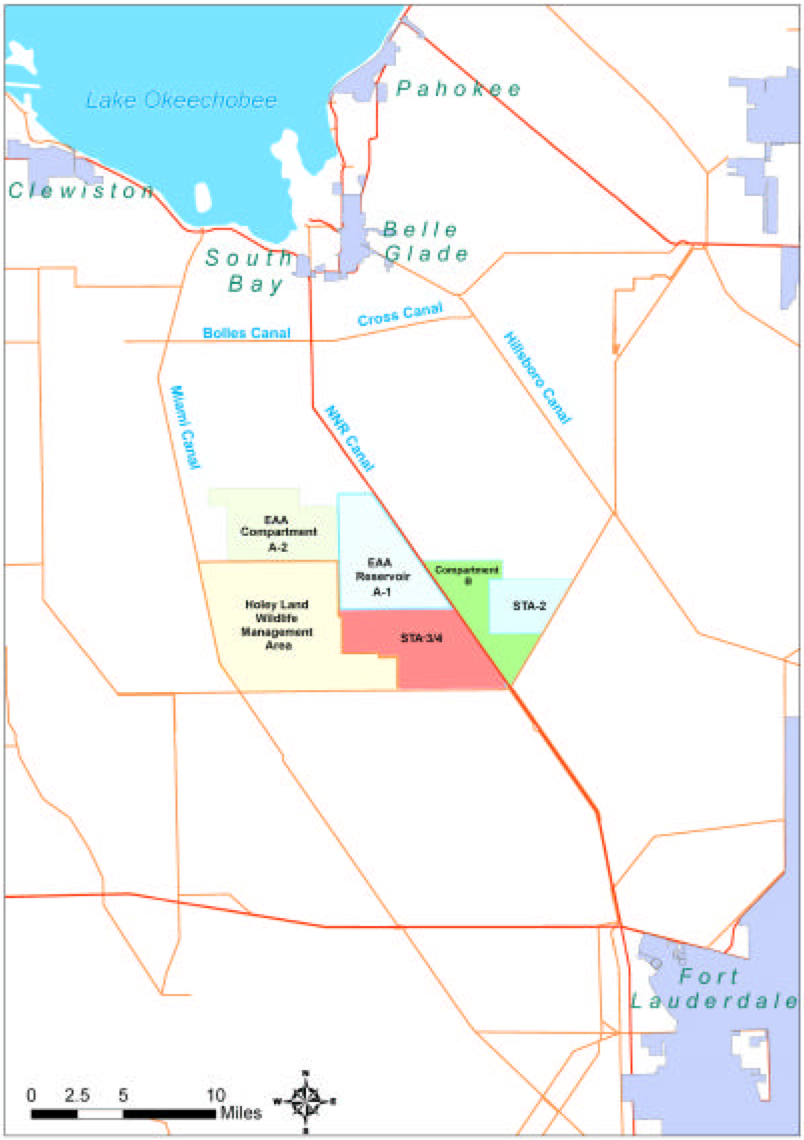
Description	Size
Total Surface Area, Acres	15,675
Length of Embankment, Feet	114,000
Total Height of Embankment, Above OG, Feet	26
Width of Embankment at Crest, Feet	14
Embankment Side Slopes	3H:1V
Elevation Bottom of EAA Reservoir A-1, Feet	8.6 NAVD88
Normal Maximum Operating Depth, Feet	12.0
Total Storage Capacity, Acre-Feet	190,000 (approximately)

20.1.1 Northeast Pump Station

A new northeast pump station will be constructed near the northeast corner of the EAA Reservoir A-1 to pump water from the NNRC to the EAA Reservoir A-1. The pump station is expected to have six pumps sized for total pumping capacity of 3,600 cfs at full design water level. Some of the pumps will be variable speed to allow them to more closely match the flows in the NNRC to maintain desired canal water level and improve operating efficiency. Design data for this structure will be determined during the 30 percent design phase.

Seepage control pumps will be included in the design of the northeast pump station. Seepage canals along the east and north sides of the EAA Reservoir A-1 will drain to the northeast pump station locations. The seepage pumps will discharge back into the EAA Reservoir A-1.

Figure 20.1-1 EAA Reservoir A-1 Location Map



20.1.2 Modifications to G-370 and G-372 Pump Stations

For purposes of this draft POM, it has been assumed that G-370 and G-372 pump stations will not be modified to deliver flow to the normal maximum operating pool of the EAA Reservoir A-1. (See Tables 20.1-2 and 20.1-3 for pump and hydraulic description of G-370 and G-372 pump stations) Several pumping alternatives are being considered that would require modifications to one or both of the pump stations, which would result in modifications to the operation scenarios presented in this document.

Table 20.1-2 Pump and Hydraulic Description of G-370 Pump Station

Pump Station Description		Other Notes
Number of Pumps	3	Inflow pumps
Discharge Capacity (each pump)	925 cfs	Pool-to-pool head 7.0 feet. Brake horsepower (Hp) 1182
Design Headwater Elevation	8.6 feet NAVD88	Headwater varies from 6.6 feet to 12.6 feet NAVD88
Headwater (HW) Start Up Condition	6.6 feet NAVD88	50 percent flow @ 23 feet head without vacuum system
Design Low Water (HW) Elevation	6.6 feet NAVD88	In front of trash screen
Maximum High water (HW) Elevation	12.6 feet NAVD88	
Maximum Screen Loss to Tailwater At Elevation 16.6 NAVD88	3.6 feet NAVD88	50 percent flow @ 13 feet head, brake Hp @ 1,315
Design tailwater elevation	13.6 feet NAVD88	
Maximum Tailwater Elevation	16.6 feet NAVD88	
Minimum Tailwater Elevation	13.1 feet NAVD88	
Nominal Pump Operation Speed	113 rpm	
Nominal “On Elevation”	As needed to maintain headwater at or below 8.6 feet NAVD88	
Nominal “off elevation”	As needed to maintain headwater at or below 8.6 feet NAVD88	
Motor/Engine Size	935 Hp	Brake horsepower @ rated condition
Motor/Engine Size	1467 Hp	Brake horsepower @ start-up condition
Motor/Engine Speed	720 rpm	Naturally aspirated 2-cycle diesel engine
Centerline Discharge Connection	22.6 feet NAVD88	Discharge sill elevation @ 18.6 feet NAVD88
Pump Station Floor Elevation	29.6 feet NAVD88	
Intake Floor Elevation	-9.9 feet NAVD88	At entrance to Formed Suction Intake tunnel
Discharge Floor Elevation	-5.4 feet NAVD88	At exit of discharge, tunnel height 12 feet

Table 20.1-3 Pump and Hydraulic Description of G-372 Pump Station

Pump Station Description		Other Notes
Number Of Pumps	4	Inflow pumps
Discharge Capacity (Each Pump)	925 cfs	Pool-to-pool head 9.0 feet. Brake Horsepower (Hp) 1182
Design Headwater Elevation	8.6 feet NAVD88	Headwater varies +6.6 to + 12.6 feet NAVD88
Design Low Water (Headwater) Elevation	6.6 feet NAVD88	Headwater level in front of screen
Start-Up Headwater Elevation	6.6 feet NAVD88	50 percent flow at 16.0 feet pool-to-pool Hd.
Maximum High Water Headwater Elevation	12.6 feet NAVD88	
Maximum Screen Loss Headwater Elevation	3.6 feet NAVD88	50 percent flow with tailwater elevation 17.6 feet NAVD88
Design Tailwater Elevation	15.6 feet NAVD88	
Maximum Tailwater Elevation	17.6 feet NAVD88	
Minimum Tailwater Elevation	13.1 feet NAVD88	
Nominal Pump Operation Speed	119 rpm	
Nominal "On Elevation"	As needed to maintain headwater at or below 8.6 feet NAVD88	
Nominal "Off Elevation"	As needed to maintain headwater at or below 8.6 feet NAVD88	
Motor/Engine Size	1663 Hp	Start-up condition
Motor/Engine Speed	720 rpm	2 cycle diesel naturally aspirated
Centerline Discharge Connection	22.6 feet NAVD88	Discharge sill elevation at 18.6 feet NAVD88
Pump Station Floor Elevation	29.6 feet NAVD88	
Intake Floor Elevation	-9.9 feet NAVD88	At FSI tunnel entrance
Discharge Floor Elevation	-5.4 feet NAVD88	At exit of discharge, tunnel height 12 feet

It is acknowledged that previous studies have included the recommendation that in order to optimize the treatment performance of the linked EAA Reservoir A-1 and STA-3/4, when feasible, all inflows to STA-3/4 should first be routed through the EAA Reservoir A-1. The operating scheme described in this draft POM, however, does not anticipate routing all flows through the EAA Reservoir A-1 prior to discharging to STA-3/4. Particularly, during periods when G-370 and G-372 pump stations are being utilized to fill the EAA Reservoir A-1, any water needed for maintaining the desired STA-3/4 operating water level would not pass through the EAA Reservoir A-1. Additional control structures would be required if the SFWMD concludes that facilities should be constructed such that all water entering the STA-3/4 could first pass through the EAA Reservoir A-1. It should be noted that higher operating costs would also result from a policy of routing all flow through the EAA Reservoir A-1. Regardless of the eventual direction on this issue, flexibility will be maintained to bypass the EAA Reservoir A-1, and deliver water directly to STA-3/4 from the NNRC and Miami Canal.

20.1.3 EAA Reservoir A-1 Gate Structures

Two EAA Reservoir A-1 gate structures will be provided for discharge from the EAA Reservoir A-1 to the Inflow and Supply Canals feeding the STA-3/4. (see Table 20.1-4 for Inflow/Supply Canal, Levee, Hydraulic Parameters, and Figure 20.1-2 for the EAA Reservoir A-1 Control Structures Location Map). One structure (southeast gate) will be located in the south EAA Reservoir A-1 embankment between G-370 pump station and control structure G-383. The second structure (southwest gate) will be placed on the west side of the EAA Reservoir A-1 at the location where the Supply Canal turns south and parallels the EAA Reservoir A-1 embankment before joining the inflow canal.

Figure 20.1-2 EAA Reservoir A-1 Control Structures Location Map

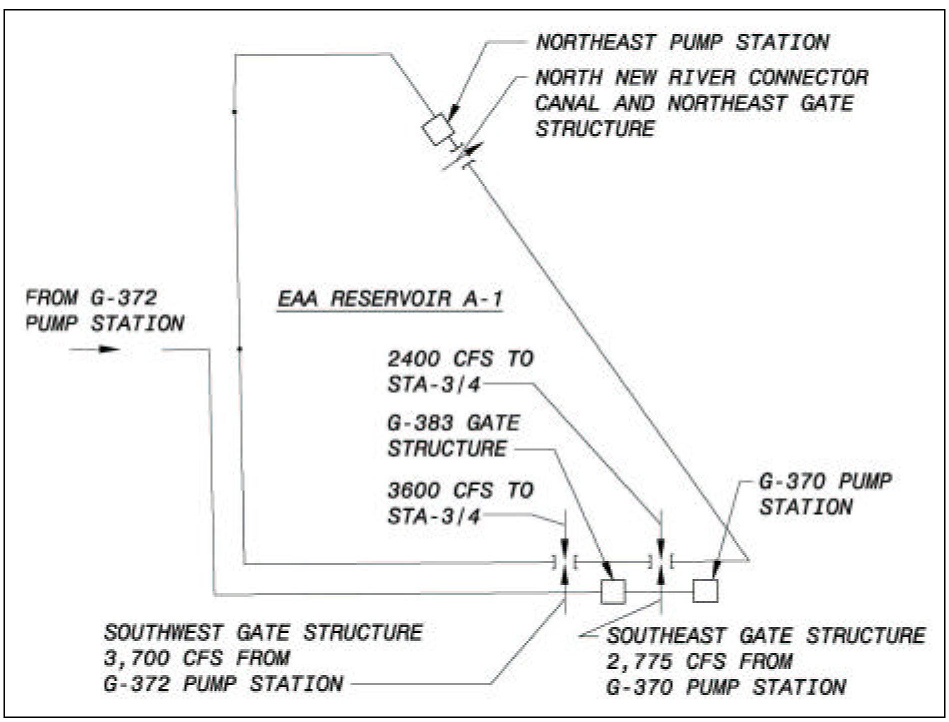


Table 20.1-4 Inflow/Supply Canal Levee Hydraulic Parameters

Canal Description	Inflow Canal[1]	Supply Canal
Canal Length	6.2 miles	10.4 miles
Canal Invert	-6.9 feet NAVD88	-6.9 feet NAVD88
Canal Bottom Width	30 to 45 feet	45 feet
Canal Side Slopes	2.5H:1V	2.5H:1V
Exterior Embankment Height	17.6 feet NAVD88	21.6 feet NAVD88
Holey Land Embankment Height	-	20.6 feet NAVD88
Inflow Control Embankment Height	18.1 feet NAVD88	-
Berm Heights	12.6 feet NAVD88	12.6 feet NAVD88
Design Maximum Flow	2,775 cfs[2]	3,670 cfs
Design Water Surface Elevation	13.6 feet NAVD88	15.6 (13.6) feet NAVD88
Design Maximum Canal Velocities	0.29 to 1.71 fps[3]	0.32 to 1.88 fps
Standard Project Storm Flow	2,775 cfs	3,670 cfs
Standard Project Storm Water Surface Elevation	16.6 feet NAVD88	18.6 (16.6) feet NAVD88
Standard Project Storm Canal Velocities	<Design Velocity	<Design Velocity

[1]The original designer distinguished between two canal sections. The portion adjacent to STA-3/4 was designated Inflow Canal while the portion abutting the Holey Land on the north and west sides of the Holey Land were designated Supply Canal. At the present time, both canal reaches are commonly referred to as the Supply Canal.
*Inflow Canal Section from G-380F to G-383 Gate Structures
[2] cfs = cubic feet per second
[3]fps = feet per second

The two gate structures will be multiple-barreled, gated concrete box culverts to allow flow into the EAA Reservoir A-1 from the inflow canal and out of the EAA Reservoir A-1 to the canals, depending on the water level and operation of the STA-3/4. Data for these structures will be developed during the design phase (see Figure 20.1-3 for a schematic of STA-3/4 structures).

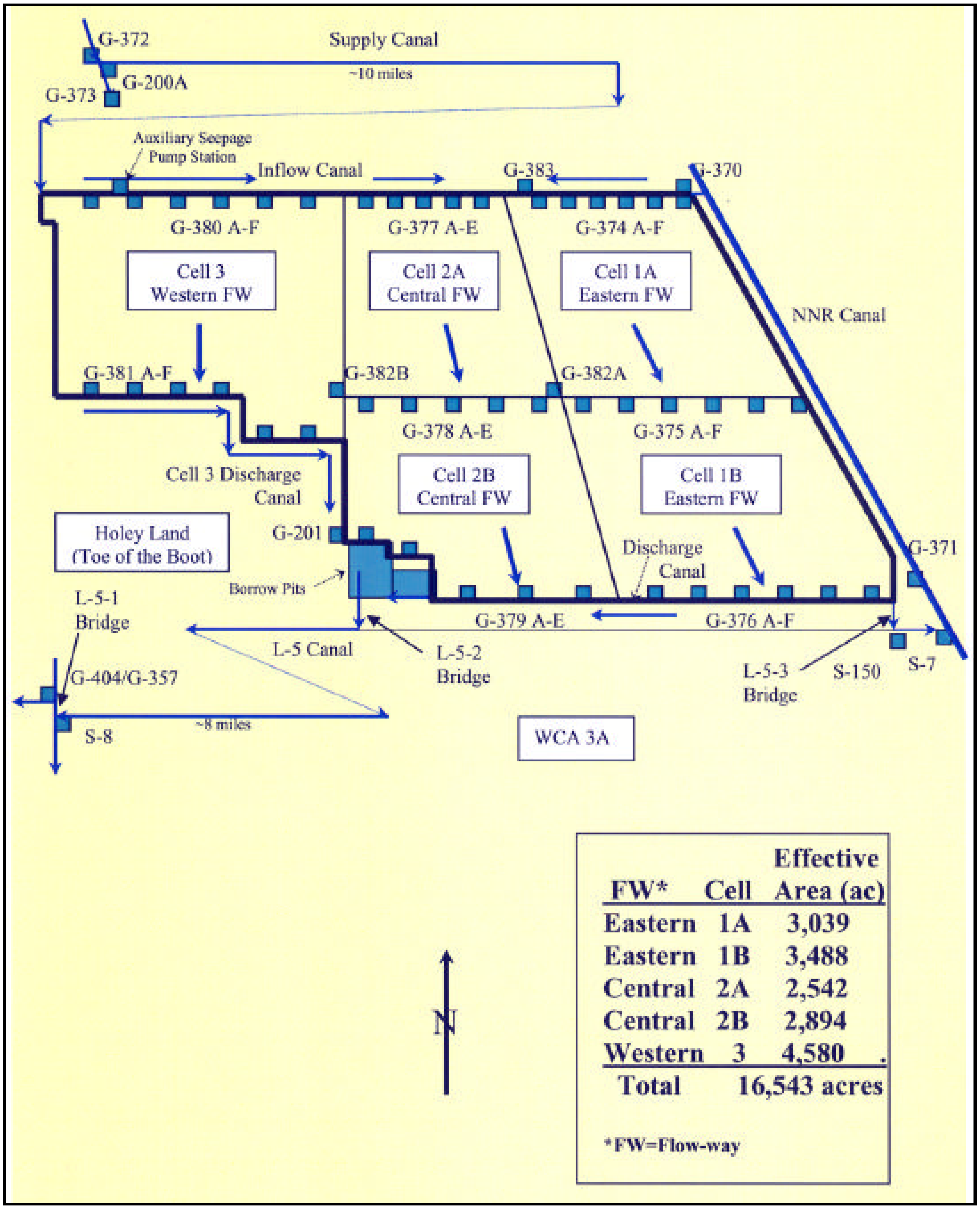
20.1.4 Northeast Gate Structure

A new gate structure will be constructed near the northeast pump station. Its primary use will be for releasing water to the NNRC for agricultural deliveries. The gates will be available for opening in an emergency. Incorporated into this structure will be an orifice type overflow. The structure will connect to the new connector canal between the northeast pump station and the NNRC.

20.1.5 Spillway

An uncontrolled orifice type spillway will be integral with the northeast gate structure near the northeast pump station. The crest will be set at an elevation of 20.6 feet NAVD88 and the orifice will be sized to limit discharges to 20 cfs per square mile (CSM) with a depth of flow over the crest of two feet. The spillway will discharge to the headwater canal for the northeast pump station and discharges will flow to the NNRC.

Figure 20.1-3 Schematic of STA-3/4 Structures



(Not to scale)

20.1.6 Removed Features

20.1.6.1 Auxiliary Seepage Pump Station

Existing facilities for an auxiliary pump station located in the western section of the Inflow Canal near control structure G-380E include two-42 inch diameter steel discharge pipes and an equipment pad for installation of SFWMD furnished portable hydraulic pumps. It was expected that this pump station could be activated if seepage rates from the Inflow Canal exceeded the capability of the G-370 and G-372 pump stations to maintain desired seepage canal levels. The existing seepage canal in this area will be eliminated with the construction of EAA Reservoir A-1, and therefore, these facilities will no longer be needed (see Tables 20.1-5 and 20.1-6 for specifics about inflow control structures G-374, G-377, G-380, and G-383, respectively).

Table 20.1-5 Inflow Control Structures G-374, G-377, and G-380

Control Structure Description	G-374 A-F[1]	G-377 A-E	G-380 A-F
Number of Culverts	6	5	6
Culvert / Gate Size (H x W)	8 feet x 10 feet	9 feet x 10 feet	7 feet x 7 feet
Culvert Length (including wingwalls)	115 feet	115 feet	111 feet
Culvert Invert	0.6 feet NAVD88	-0.4 feet NAVD88	1.6 feet NAVD88
Design Inflow (each culvert)	362 cfs	396 cfs	282 cfs
Design Maximum Headwater Elevation	13.5 feet NAVD88	13.5 feet NAVD88	13.7 feet NAVD88
Design Low Water (Headwater) Elevation	9.6 feet NAVD88	9.6 feet NAVD88	9.6 feet NAVD88
Standard Project Storm (Headwater) Elevation	16.6 NAVD88	16.6 feet NAVD88	16.6 feet NAVD88
Design Maximum Tailwater Elevation	13.0 feet NAVD88	13.0 feet NAVD88	12.6 feet NAVD88
Design Low Water (Tailwater) Elevation	9.2 feet NAVD88	9.5 feet NAVD88	9.6 feet NAVD88
Standard Project Storm (Tailwater) Elevation	16.0 feet NAVD88	16.0 feet NAVD88	15.6 feet NAVD88
Headwater/Tailwater Data Via Telemetry	G-374 B & E	G-377 B & D	G-380 B & E

[1]Control Structure Descriptions are illustrated in Figure 20.1-3
cfs = cubic feet per second

Table 20.1-6 Control Structure G-383

Control Structure Description	G-383
Number of Culverts	2
Culvert / Gate Size (H x W)	10 feet x 10 feet
Culvert Length (Including Wingwalls)	114 feet
Culvert Invert	-1.4 feet NAVD88
Design Inflow (Each Culvert)	735 cfs[1]
Design Maximum Water Elevation	13.6 feet NAVD88
Design Low Water Elevation	9.6 feet NAVD88
Standard Project Storm Elevation	16.6 feet NAVD88
Maximum Differential Head	1.4 feet
Headwater/Tailwater Data Via Telemetry	G-374 E and G-377 B

[1]cfs = cubic feet per second

20.1.7 Operational Interaction of Project Features

EAA Reservoir A-1 will store excess stormwater that can be sent to STA-3/4 at a later time thereby improving the quantity, timing, and distribution of water dedicated and managed for the natural system. In addition, storage of storm runoff will reduce flooding and provide water, which would otherwise have passed to tide, that can be released for agricultural purposes.

Operations for STA-3/4 will change when the EAA Reservoir A-1 is completed. Currently the only sources of water for STA-3/4 are through G-370 and G-372 pump stations, and delivery rates to the environment through the STA-3/4 are completely dependent upon immediate water availability in the NNRC and Miami Canal. During periods of high runoff, the STA-3/4 experiences surges that stress the viability of the treatment system. Storm surges will be dampened by EAA Reservoir A-1, and the options available to SFWMD will increase significantly. Releases from Lake Okeechobee will be delivered to the EAA Reservoir A-1 and STA-3/4 system during periods of low or no runoff from the EAA.

Factors which will impact operating decisions include:

- Water level in the EAA Reservoir A-1
- Lake Okeechobee water level
- Availability of water in the NNRC and Miami Canal
- Desired operating level in STA-3/4
- Impending or existing weather conditions
- Environmental deliveries needed
- Agricultural deliveries needed
- Water availability from other watersheds

To some extent, the time of year may also impact decisions regarding the operation of the EAA Reservoir A-1. For example, it may be desirable during the wet season when EAA Reservoir A-1 is full, or above a predetermined water level, to increase deliveries to STA-3/4 in order to maintain storage capacity for excess runoff. These deliveries would only be sent to STA-3/4 if sufficient hydraulic and treatment capacity exists. In this manner, flexibility to capture excessive runoff would be enhanced and flooding of farms would be decreased. This practice should also decrease the volume and frequency of back pumping to Lake Okeechobee. Near the end of the hurricane season, water levels in EAA Reservoir A-1 would be allowed to increase to the full level in order to take full advantage of its storage capacity and to improve capability for supplying environmental and agricultural deliveries during dry times.

Using the recommended alternatives for pumping and control structures, operating scenarios are shown below to demonstrate some of the options that would be available to SFWMD.

- Empty EAA Reservoir A-1, Water Available in the Primary Canals, and no Water Deliveries Needed for Environment or Agriculture
    - G-370 and G-372 pump stations delivering water to EAA Reservoir A-1 via the Inflow and Supply Canals

  - o Northeast pump station delivering water to the EAA Reservoir A-1 if excess water is available in the NNRC. Note that, depending upon the available water in the NNRC, SWFMD may choose to shut down G-370 pump station and use only the northeast pump station if it results in more efficient operations

- Empty EAA Reservoir A-1, Water Available in the Primary Canals, and Environmental Water Deliveries Desired
  - o G-370 and G-372 pump stations supplying water for delivery to the environment by pumping into the Inflow and Supply Canals
  - o Water from the Inflow and Supply Canals in excess of that being supplied to the environment overflows to EAA Reservoir A-1
  - o Northeast pump station supplies EAA Reservoir A-1 if excess water is available in the NNRC

- EAA Reservoir A-1 Level above Maximum Water Level in STA-3/4 Supply Canal, Water Available in Primary Canals, and Environmental Water Deliveries Desired

  Option 1
  - o G-370 and G-372 pump stations delivering water to the environment by pumping into the Supply Canal if water is available from the NNRC or the Miami canal
  - o Northeast pump station supplies EAA Reservoir A-1 if excess water is available from the NNRC
  - o EAA Reservoir A-1 supplies water to the Supply Canal, if sufficient water is not available from the NNRC or the Miami Canal

  Option 2
  - o If sufficient water is available from the Miami Canal, G-370 pump station is shut down and G-372 pump station supplies water to STA-3/4 by pumping into the Supply Canal
  - o Northeast pump station delivers excess water for the NRNC to the EAA Reservoir A-1
  - o EAA Reservoir A-1 supplies water to the Supply Canal, if necessary

  Option 3
  - o G-370 and G-372 pump stations are shut down
  - o Northeast pump station supplies water to the EAA Reservoir A-1
  - o EAA Reservoir A-1 supplies water to the Supply Canal for environmental deliveries

- EAA Reservoir A-1 Full, Water Available in Primary Canals, and Environmental Water Deliveries Desired

Option 1

- o G-370 and G-372 pump stations deliver water to STA-3/4 by pumping into the Supply Canal
- o Northeast pump station operated as necessary to make up evaporation and seepage losses in EAA Reservoir A-1

Option 2

- o G-370 and G-372 pump stations are shut down
- o EAA Reservoir A-1 supplies water to the Supply Canal
- o Northeast pump station supplies water to the EAA Reservoir A-1

EAA Reservoir A-1 will provide water for substantial agricultural deliveries by capturing runoff that would otherwise have gone to tide. Agricultural deliveries that cannot be met by the EAA Reservoir A-1 will continue to be supplied from Lake Okeechobee. When water is available in the EAA Reservoir A-1 for agriculture deliveries, it will normally be released through the northeast gate structure located near the northeast pump station from where it will flow to the NNRC via the connector canal for the pump station. When the EAA Reservoir A-1 water level is below that needed for gravity flow to the NNRC, pumps located in the northeast pump station will be activated.

20.2 OPERATIONAL STRATEGY TO MEET PROJECT OBJECTIVES

The Draft Operating Plan for the EAA Reservoir A-1 will be modified and revised, as necessary, through several Project phases. During the detailed design phase, the Draft Operating Plan will be modified to define any temporary operations to be used during construction including startup and filling. The Operation Plan for STA-3/4 will also be modified as required to reflect operations during periods when construction along and within the embankments for the inflow and Supply Canals could disrupt operations.

Knowledge gained from the Operational Testing and Monitoring Phase will then be incorporated into the Draft Operating Plan, which will be coordinated with SFWMD and the USACE South Atlantic Division (SAD), and will supersede all other iterations of the Draft Operating Plan. The final version of the Draft Operating Plan will be used by SFWMD when they accept responsibility for long-term operations of the EAA Reservoir A-1.

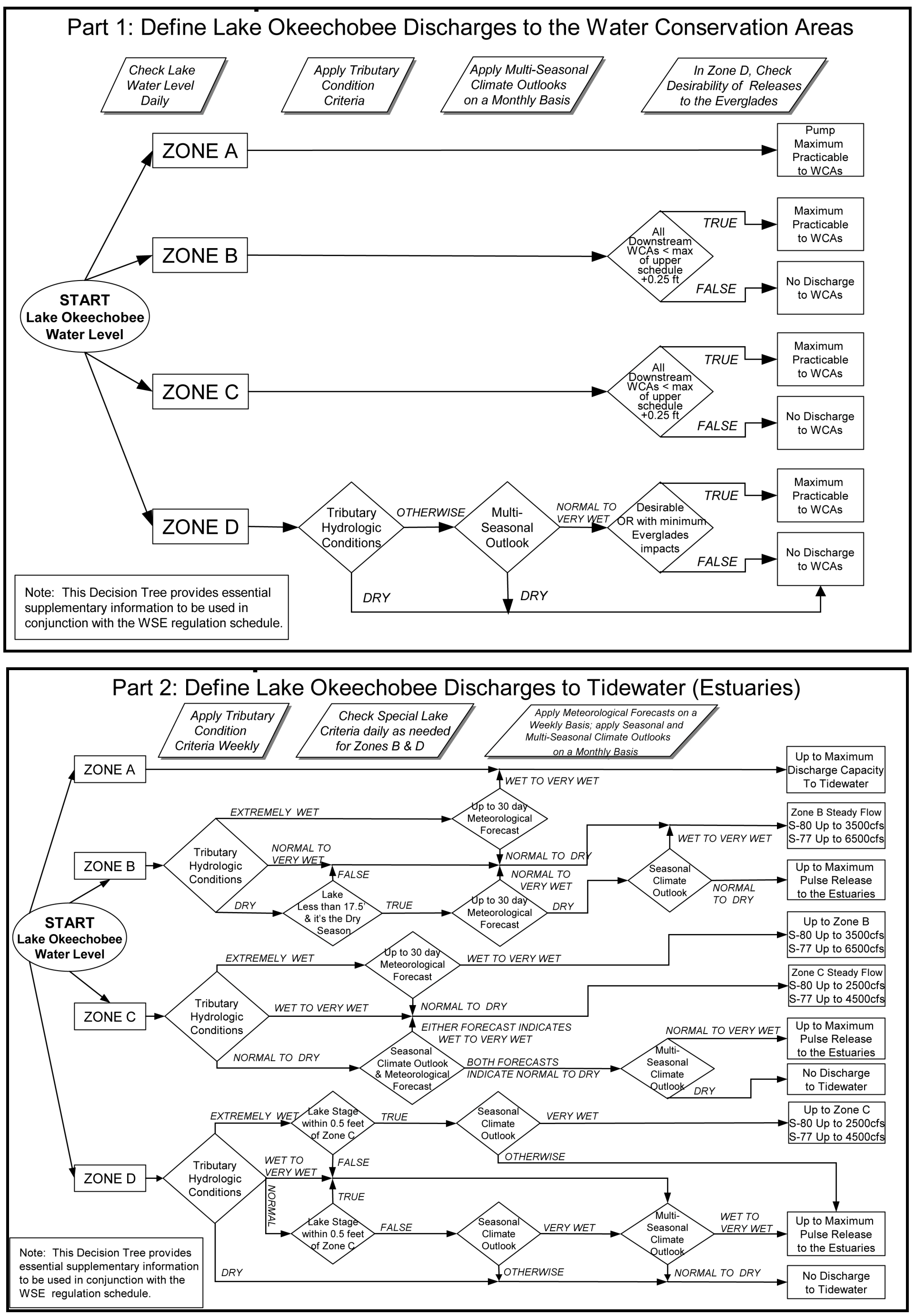
The current Lake Okeechobee regulation schedule indicates that when the Lake elevation is in zones A, B, or C, releases are made per the USACE's WSE Decision Tree (Figure 20.2-1). The construction of EAA Reservoir A-1 will allow Lake Okeechobee regulation discharges to be released to EAA Reservoir A-1 when storage is available, rather than to the estuaries of the Caloosahatchee and St. Lucie Rivers. During wet conditions, runoff captured by the NNRC and Miami Canal will be stored in the EAA Reservoir A-1. This stored water will be used to supplement agricultural water use in the NNRC basin, and to deliver water to the environment. The need to back pump water to Lake Okeechobee will also be reduced and overall flood protection will be enhanced.

20.3 PROJECT RELATIONSHIPS AND INTERACTIONS

Operation of the EAA Reservoir A-1 and associated structures is linked to the operation of STA-3/4. Before the new facilities are in place, some modifications to the Operating Plan for STA-3/4 will be required to incorporate the EAA Reservoir A-1's storage capability for dry weather releases and for potential decreased stormwater flows to STA-3/4 during EAA Reservoir A-1 filling operations.

Other systems downstream from the STA-3/4, including the WCA-3A Everglades Protection Area may also impact operation of the EAA Reservoir A-1 system.

Figure 20.2-1 WSE Operational Guidelines Decision Trees



source: http://www.saj.usace.army.mil/h2o/lib/documents/WSE/wsedectree.pdf

20.4 MAJOR CONSTRAINTS

Constraints to the operation of the EAA Reservoir A-1 system include the availability of water in the NNRC and Miami Canal, water availability from Lake Okeechobee, the requirement of maintaining a minimum water elevation in the inflow canal to maintain minimum stages in STA-3/4 cells, and the varying agricultural deliveries.

During drought conditions, sufficient water may not be available to completely fill the EAA Reservoir A-1 on an annual basis, resulting in potential decreased capacity to maintain environmental deliveries and agricultural deliveries during the dry season.

Since pumping to the EAA Reservoir A-1 will occur mostly during the wet season, (Lake Okeechobee regulatory releases may also be sent to the EAA Reservoir A-1 during the wet season), operation of STA-3/4 during those periods will dictate how much of the total canal flow will be available for storage. Likewise, environmental and agricultural deliveries may conflict, and therefore, constrain the distribution of the stored water for those purposes.

Minimum operating levels for both the Supply Canal and NNRC will prevent gravity releases from the EAA Reservoir A-1 when the EAA Reservoir A-1 operating level is low.

20.5 STANDING INSTRUCTIONS TO PROJECT OPERATORS

Once the operational testing and monitoring phase of components of EAA Reservoir A-1 has been completed, SFWMD will be responsible for the day-to-day water management operations. During normal conditions, the EAA Reservoir A-1 water control structures shall be operated in accordance with the approved Operating Plan for the EAA Reservoir A-1. Standing instructions will be drafted during the detailed design phase and finalized during the construction phase.

20.6 OPERATIONS TO MEET PROJECT PURPOSES

20.6.1 Achieving Natural System Goals, Objectives, and Benefits

Currently, when the Lake Okeechobee elevation is in zone A, B, C, or D (as illustrated in Figure 20.1-3), regulatory releases are made through the St. Lucie Canal and the Caloosahatchee River which flow to estuaries downstream. These releases from Lake Okeechobee have resulted in declines in aquatic vegetation and oyster populations. Upon completion of EAA Reservoir A-1, a portion of the flow that would otherwise have gone to the St. Lucie Canal and the Caloosahatchee River will be sent to EAA Reservoir A-1. When water levels in the primary canals reach predetermined levels, the G-370 and G-372 pump stations, and the new northeast pump station will be operated to pump the released waters to the EAA Reservoir A-1. Stored water can later be released to the Supply Canal for environmental deliveries when the EAA Reservoir A-1 water level exceeds the minimum Supply Canal water level. When the EAA Reservoir A-1 water level is below the minimum water level for the Supply Canal, water may be released through the northeast gate structure into the NNRC from where it can be lifted into the Supply Canal by G-370 pump station. When appropriate, G-370 and G-372 pump stations can also pump Lake Okeechobee releases directly to the inflow and Supply Canals for environmental deliveries.

20.7 FLOOD DAMAGE REDUCTION

20.7.1 Normal and Emergency Operations

When the NNRC and Miami Canal are above predetermined levels, G-370 and G-372 pump stations will pump water into the Inflow and Supply Canals for storage in the EAA Reservoir A-1. The northeast pump station can operate in conjunction with or in lieu of the G-370 pump station to deliver water from the NNRC to EAA Reservoir A-1. If EAA Reservoir A-1 is at the normal full level, G-370 and G-372 pump stations can be operated to deliver water directly to the environment by pumping to the inflow and Supply Canals.

20.7.2 Hurricane or Tropical Storm Operations

The hurricane season occurs each year from June 1 through November 30. When there are tropical depressions, tropical storms, and/or hurricanes in the Atlantic/Caribbean Basin or the Gulf Coast of Florida, the National Hurricane Center issues public advisories, forecast advisories, forecast discussions, and strike probability forecasts.

Water management operations within the EAA Reservoir A-1 during hurricanes or tropical storms should follow SFWMD Emergency Preparedness Manual Suggested Hurricane Operation Procedures, April 2004. The USACE, Jacksonville District, Emergency Operations Standard Operating Procedures document (CESAJ SOP 500-1-1) should be consulted for emergency preparation and actions.

20.7.3 Reservoir Emergency Overflow/Uncontrolled Discharge

An uncontrolled orifice type spillway will be constructed as described in this Operating Manual under Proposed Features, including any required provisions for operating the EAA Reservoir A-1 to avoid re-suspension of phosphorus.

20.8 WATER QUALITY

Additional operational procedures to improve water quality will be developed during the detailed design phase of the EAA Reservoir A-1 Project and will be included in the final Operating Manual. This includes any required operating provisions to avoid resuspension of phosphorus.

20.9 WATER SUPPLY OPERATIONS

During dry conditions when water is needed for agricultural deliveries, and the EAA Reservoir A-1 level is above 11.5 feet NAVD88, the northeast gate structure can be opened as necessary to allow gravity discharge to supply agricultural deliveries to the NNRC. When the EAA Reservoir A-1 water level is below that elevation, provisions will be available the northeast pump station to pump water to the NNRC.

If environmental deliveries are needed and the water level in the EAA Reservoir A-1 is above the water level in the Supply Canal, water can be released through southeast and southwest gates. If the EAA Reservoir A-1 water level is less than the required water level in the Supply Canal, water can be released through the northeast gate structure near the northeast pump station, or pumped from the northeast pump station to the NNRC and then pumped by G-370 gate structure from the NNRC into the Supply Canal. G-383 pump station would be opened to allow flow to the western flow ways.

20.10 RECREATION

Activities such as fishing and boating will be permitted at the discretion of the SFWMD. No special operations will be required.

20.11 FISH AND WILDLIFE

Existing canals within the EAA Reservoir A-1 site, along with borrow canals and quarry areas, will provide deep-water refugia. In addition, littoral shelves will be incorporated along the seepage canal. No special operations will be required.

20.12 PRESTORM/STORM OPERATION

If there is unused storage capacity in the EAA Reservoir A-1, the preferred operating mode will be to maximize pumping into the EAA Reservoir A-1 during storm events. This operation would decrease the impact of high flow stormwater events on STA-3/4. If the northeast pump station is operating to full capacity and the NNRC has excel flow, G-370 pump station will be activated to pump into the EAA Reservoir A-1 or directly into STA-3/4.

If a heavy rainfall is forecasted by the National Weather Service Advisories and SFWMD, a pre-storm drawdown of EAA Reservoir A-1 may be initiated to increase available storage capacity. Storage may be created by discharging to the NNRC through the northeast gate structure or to the Inflow Canal through the southeast and southwest gate structures. The ability to discharge to either the environment or to the NNRC will be a function of the water depths and flows at the time.

If the EAA Reservoir A-1 exceeds the normal maximum operating pool as the result of a storm, operations would include drawdown of the EAA Reservoir A-1 by releasing water to STA-3/4 and/or NNRC in order to bring the water level back to the normal maximum operating pool.

20.13 CONSISTENCY WITH THE IDENTIFICATION OF WATER RESERVATIONS OR ALLOCATIONS FOR THE NATURAL SYSTEM

Certain EAA Reservoir A-1 Project assurances analyses are not yet complete. This section will be updated during the detailed design phase. The appropriate quantity, timing and distribution of water for the natural system and other water related needs will be identified in the PIR.

The EAA Reservoir A-1 will store runoff that would otherwise have gone to tide and will improve the timing and distribution of water deliveries to the environment. It has been demonstrated using an area specific computer model, and POS data from the SFWMM (which is the same as the EPC 2010 and 2015 version 5.4.2), that more than 600,000 acre-feet per year can be delivered to the environment by EAA Reservoir A-1. Operating criteria for EAA Reservoir A-1 will be developed in subsequent versions of this manual to be consistent with the water reservations or allocations for the natural system made by the State in accordance with Section 373.1501(5) F.S.

20.14 CONSISTENCY WITH SAVING CLAUSE AND STATE ASSURANCES PROVISIONS

During periods when EAA Reservoir A-1 contains water and it is necessary to prevent seepage from impacting adjacent properties, the seepage canal water level can be pumped down as required to prevent the groundwater level from rising. A groundwater model has been utilized to verify that depressing the seepage canal will be effective in preventing flooding of adjacent properties. Other alternatives have been considered. An alternative will be selected for implementation prior to initiation of the 30 percent design.

The EAA Reservoir A-1 will provide capacity for storage of storm runoff and will increase the pumping capacity from the NNRC. In addition, the area occupied by the EAA Reservoir A-1 previously used for agriculture will no longer deliver runoff to the NNRC, thereby making available 500 cfs of NNRC capacity that was previously unavailable. Therefore, the Project will not diminish flood protection and should reduce flooding in the NNRC under most conditions.

The Project will store runoff that would otherwise have gone to tide and will, therefore, provide water for agricultural uses during the dry season. It has been demonstrated using an area specific computer model that a high percentage of the agricultural deliveries along the NNRC can be provided by EAA Reservoir A-1.

A berm will be constructed outside of the seepage canal and any runoff between the berm and the EAA Reservoir A-1 embankment will be collected in the seepage canal and delivered to the EAA Reservoir A-1.

20.15 DROUGHT CONTINGENCY PLAN

During a drought in the EAA Reservoir A-1 Project area, operations will be in accordance with SFWMD Rules, Chapter 40E-21, F.A.C., Water Shortage Plan.

20.16 FLOOD EMERGENCY ACTION PLAN

The Flood Emergency Action Plan will be completed for the EAA Reservoir A-1 prior to completion of construction. The Flood Emergency Action Plan to be developed should be consulted for related emergency preparation and action. Local emergency management offices will be provided copies of the Flood Emergency Action Plan as necessary. This plan may be used to supplement Hurricane or Tropical Storm Regulations. As outlined in USACE Engineering Regulation 1130-2-530, the Flood Emergency Action Plan shall include:

- A written Emergency Notification Procedure for serious abnormal conditions to provide for safety of people in the vicinity of the EAA Reservoir A-1 area and also to trigger immediate response for remedial assistance to the embankment and water control structures
- A description or list of conditions leading to emergency situations and ways of dealing with them should they occur
- Storage area dewatering procedures
- Embankment and water control structure failure inundation maps
- Listing of location, types, and quantity of emergency repair materials and equipment
- Details outlining responsibilities for inspection and execution of emergency repairs
- List of contractors available within a reasonable distance of the EAA Reservoir A-1

20.17 DEVIATIONS FROM NORMAL OPERATING CRITERIA

The USACE District Commander is occasionally requested by the non-Federal sponsor to approve deviations from normal operation criteria. Prior approval for a deviation is required from USACE-SAD except as noted in paragraphs below. Deviation requests usually fall into the following categories:

20.17.1 Emergencies

Emergencies that can be expected include water recreation related accidents such as drowning or boating accidents, failure of EAA Reservoir A-1 facilities, and flushing of pollutants. Water control actions necessary to abate the problems should be implemented immediately unless such action would create worse conditions. The USACE-SAD office must be informed of the problem and the emergency operating changes as soon as practical. In addition, the non-Federal sponsor, the State of Florida (Florida Department of Environmental Protection and SFWMD), and the U.S. Department of the Interior should be informed.

20.17.2 Unplanned Minor Deviations

There are unplanned instances that create a temporary need for minor deviations from the normal operating criteria, although they are not considered emergencies. Construction accounts for the major portion of incidents requiring minor deviations. Deviations are also sometimes necessary to carry out the maintenance and inspection of facilities. Request for changes in release rates generally involve time periods ranging from a few days to a few weeks. Each request should be analyzed on its own merits. In evaluating the proposed deviation, consideration must be given to low flow requirements, fish and wildlife, water rights, roles of the USACE and the SFWMD, short-term release scheduling, long-range release planning, and storage utilization (seasonal commingled, joint use).

20.18 SEEPAGE CONTROL

The EAA Reservoir A-1 will be constructed immediately north of the existing Supply Canal. The embankment for the EAA Reservoir A-1 will be constructed over the existing seepage canal along the north side of the Supply Canal. Therefore, the existing seepage pumps in G-370 pump station will not serve their original purpose but may be connected to the seepage canal along the east side of EAA Reservoir A-1.

New seepage canals will be constructed along the northern, western, and eastern sides of the EAA Reservoir A-1 and will convey seepage to the new northeast pump station. Seepage pumps in that facility will be designed to pump the seepage flow back into the EAA Reservoir A-1.

20.19 INITIAL RESERVOIR/TREATMENT AREA FILLING PLAN

The Initial Storage Filling Plan (ISFP) is defined as a deliberate impoundment of water to meet Project purposes and is a continuing process as successively higher water levels are attained. The initial EAA Reservoir A-1 filling is the first opportunity to test whether the containing embankments and water control structures will perform as designed. To monitor this performance, the rate of filling will be controlled to the extent feasible to allow as much time as needed for implementation of a predetermined monitoring program, including the observation and analysis of instrumentation data. Information furnished in the ISFP will generally be concerned with action that can be taken without a significant impact to Project purposes, provided no unsafe conditions are observed. An ISFP will be developed during design and construction. The ISFP will include but is not limited to the following:

- Preferred filling rate and the available options to control the rate of filling, as well as the consequences of operation with the prime objective of controlling the rate of EAA Reservoir A-1 water level rise
- The most likely type of problem(s) that may develop during initial filling and the monitoring necessary to detect those problems
- A description of the proposed hydrologic data collection and transmission system, and a plan for reading the instruments and evaluating the data with regard to the filling plan
- A plan for inspecting the embankment and downstream areas prior to and during filling, including the relationship between frequency of inspection and rate of pool rise
- Instructions for observers on conditions that require immediate attention of personnel authorized to make emergency decisions. Clearly identify who is responsible for decisions and how they can be contacted. Alternative decision makers should be identified
- An emergency plan listing responsibilities, name and/or positions, telephone numbers, pager numbers, and radio frequencies to be used
- Water quality requirements, if any, for the initial filling

20.20 WATER CONTROL DATA ACQUISITION SYSTEM PLAN

The remote automation components installed at the pump stations and other structures are RTU and communications channel to SFWMD control center. The access for the RTU to the control center is via field interface units (FIU). The automation components of all pump stations and structures that will eventually be operated and maintained by SFWMD will conform to SFWMD standards.

20.21 CONSISTENCY WITH THE ADAPTIVE MANAGEMENT PROGRAM AND PERIODIC CERP UPDATES

After initiation of long-term operations and maintenance of the EAA Reservoir A-1, the Operating Manual may be further modified based on operating criteria approved by the USACE and the SFWMD that results from CERP updates and/or recommendations from the adaptive assessment process as outlined in Guidance Memorandum Number 6 of the Programmatic Regulations.

SECTION 21

COST OPTIMIZATION

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TABLES

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21. COST OPTIMIZATION

21.1 REVIEW ASSUMPTIONS

The design of the Everglades Agricultural Area (EAA) Reservoir A-1 that is currently described in this Basis of Design Report (BODR) and its associated probable cost estimate as presented in the Section 23 is based upon:

- Provision of a 190,000 acre foot EAA Reservoir A-1 at a maximum operating depth of 12 feet
- Criteria established in the Acceler8 Design Criteria Memoranda (DCMs)

The current probable cost opinion exceeds budget expectations. Therefore, the South Florida Water Management District (SFWMD) has requested that the project team identify potential scale/scope modifications that could improve the cost effectiveness of the EAA Reservoir A-1.

The review of potential cost reduction measures included a review of the major project components and their associated costs. The probable costs identified in Table 21.1-1 were submitted for SFWMD review on July 29, 2005 and were the basis for this review.

Table 21.1-1 Base Design Project Cost Components – July, 2005 Submittal

Component	Construction Cost ($ millions)	Contingency at 30 percent ($ millions)	Total Cost ($ millions)
Embankment (Excavation, Processing, & Placement)	$ 252.2	$ 75.8	$ 328.2
Seepage (Canal & Cut-off)	$ 57.9	$ 17.4	$ 75.3
Northeast Pump Station	$ 49	$ 14.7	$ 63.7
Control Structures	$ 20	$ 6	$ 26
U.S. 27 Bridge	$ 5.3	$ 1.6	$ 6.9
Total	$ 384.7	$ 115.4	$ 500.1

To effect any major cost reduction, the review focused on those elements that contribute the greatest cost. Therefore, cost reduction options considered and discussed herein are as follows:

- Reduce contingency from 30 percent mandated by DCM-7 to some smaller percentage
- Reduce embankment height through the use of a concrete wavebreak for irregular waves
- Reduce design storm for wave run-up to reduce embankment height
- Revise operation plan to limit filling procedures during hurricane season to reduce embankment height
- Reconfigure the embankment perimeter to reduce overall embankment length
- Limit maximum depth of operation of EAA Reservoir A-1 to 8 feet during Phase 1 to reduce embankment height between EAA Reservoir A-1 and EAA Reservoir A-2.
- Add an uncontrolled spillway to reduce embankment height
- Reduce pump station capacity to minimum requirements
- Combinations of the above options

Implementation of any of these concepts would be at the discretion of the SFMWD during the preliminary design phase. In order to provide a more accurate representation of potential savings, the remaining probable costs in this section are based on updates to the opinion of probable costs for the various components made after the July 29, 2005 submittal. These updates were prompted by refinements to the conceptual design of each component made after the aforementioned submittal, are shown below in Table 21.1-2 below, and are discussed in detail in Section 23 of this submittal.

Table 21.1-2 Base Design Project Cost Components – October, 2005 Submittal

Component	Construction Cost ($ millions)	Contingency at 30 percent ($ millions)	Total Cost ($ millions)
Embankment (Excavation, Processing, & Placement)	$ 252.6	$ 75.9	$ 328.5
Seepage (Canal & Cut-off)	$ 57.9	$ 17.4	$ 75.3
Northeast Pump Station	$ 70.4	$ 14.1	$ 84.5
Control Structures	$ 14.6	$ 4.4	$ 19.0
U.S. 27 Bridge	$ 5.3	$ 1.6	$ 6.9
Total	$ 400.7	$ 113.2	$ 514.2

21.2 COST REDUCTION OPTIONS

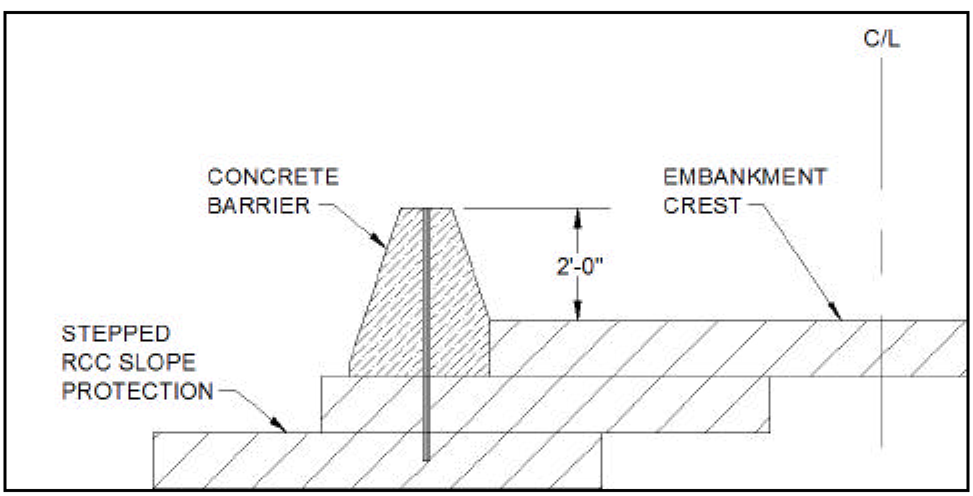
21.2.1 Reduce Embankment Height by Implementing a Concrete Parapet

The current EAA Reservoir A-1 embankment design is based on full containment, maximum calculated wind set-up and wave run-up, and includes an allowance for the irregular wave. One cost reduction measure would be to lower the embankment and include the design of a concrete parapet to contain irregular waves. Figure 21.2-1 illustrates a conceptual parapet design that incorporates a 2-ft tall parapet resulting in a 1.5-ft reduction in embankment height. This would result in a savings of approximately $10 million as shown in Table 21.2-1.

Table 21.2-1 Lowered Embankment with Parapet

Component	Construction Cost	Contingency at 30 percent	Total Cost
Reduction in Embankment Cost	$ 14,444,000	$ 4,342,000	$ 18,786,000
Cost of Parapet	$ 6,504,000	$ 1,951,000	$ 8,455,000
		Net Savings	**$ 10,331,000**

Figure 21.2-1 Full Height Embankment versus Lowered Embankment with Parapet



21.2.2 Reduce Design Storm for Wave Run-up Modeling

Three cases were analyzed for wave run-up in accordance with DCM 2 with the results shown in Table 21.2-2. Based on this analysis, the EAA Reservoir A-1 embankment height was initially established as 26 feet above grade level and was based on Case 3 conditions containing a regular wave only. The final draft of DCM-2 issued on August 19, 2005 (Haapala, 2005) states that, "Case 3 is to be evaluated and used as part of a sensitivity/parametric analysis of set-up and wave height analysis, but will not be required as the final basis for determining the minimum freeboard height." Case 3 is based on an event with a probable maximum wind (PMW) with sustained wind speeds of 158 miles per hour (mph) but with no precipitation. Wind speeds of this magnitude would be attributable to a Category 5 hurricane, but have never been recorded this far inland in the United States. Maximum estimated wind speeds from a fast moving, Category 5 hurricane (assuming a maximum sustained wind speed of 155 mph off shore) at the EAA Reservoir A-1 site is approximated at 125 mph according to the National Oceanic & Atmospheric Administration's (NOAA) "Inland Wind Model." Therefore the probability of a Case 3 event is relatively remote.

Table 21.2-2 Wave Run-up & Wind Setup Results

Case	Wind Speed (mph)	Rainfall Depth (ft)	Effective Depth (ft)	Wave Run-up (ft)	Wind Setup (ft)	Maximum Water Level (ft)	
						Regular Wave	Irregular Wave
Case 1. PMP, 100-year wind	103	4.5	16.5	6.0	2.1	24.6	25.5
Case 1a. 500 year rain, 100 year wind	103	1.7	13.7	5.45	2.45	21.6	22.5
Case 1b. Regional PMP, 100 year wind	103	3.5	15.5	5.8	2.2	23.5	24.5
Case 2. 100 year rain, Category 5 wind	122	1.4	13.4	6.1	3.6	23.1	24.0
Case 3. PMW, no rain	158	0.0	12.0	6.7	7.0	25.7	27.5
Case 3a. 500 year wind, no rain	119	0.0	12.0	5.6	3.8	21.4	22.0

Notes:
1. Maximum Water Level (MWL) is measured from the reservoir bottom (original ground level)
2. Effective Depth is the sum of normal maximum operating level (12 ft) plus rainfall and is the depth used to calculate wind set-up
3. MWL for the regular (monochromatic) wave is the sum of effective depth, wave run-up and wind set-up
4. MWL for the irregular wave is the depth above bottom at which overtopping is less than 0.1 cfs/ft, defined as zero overtopping in DCM-2

Consequently, the governing design condition is Case 1, which is based on an event with the probable maximum precipitation (PMP), defined as about 54 inches of rainfall in a 72-hour period, combined with a 100-year wind, defined as winds of 103 mph. The PMP is theoretically defined as the greatest amount of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year. To gain perspective on the magnitude of this event, a 100-year, 72-hour rainfall event would be about 15.2 inches of rainfall; a 500-year, 72-hour rainfall event would be about 19 inches of rainfall. The PMP of 54 inches of rainfall in 72 hours is estimated to have a recurrence interval that would be less than once every 10,000 years. Although Case 1 is the governing design condition, the probability of occurrence is extremely remote.

Of the three cases identified in DCM-2, Case 2, a 100-year rainfall event combined with a Category 5 wind speed of 122 mph, is the most likely to occur. Three additional cases were evaluated as follows:

- Case 1a: 100-year wind combined with a 500 year rainfall event
- Case 1b: 100-year wind combined with a regional PMP
- Case 3a: 500-year wind (normal pool level, no rainfall)

The results for these three cases are also summarized in Table 21.2-2.

The definition of Cases 1a and 3a is self evident. Case 1b relates to a regional PMP which would be a lesser amount of rainfall over a larger area than that for Case 1. In reservoir design, the PMP normally represents that precipitation that would fall on the reservoir and its drainage basin. For this situation, because the EAA Reservoir A-1 is off-line, its surface area represents the entire drainage area. Therefore, a PMP represents a deluge that would be centered entirely over the EAA Reservoir A-1 itself, while surrounding areas receive lesser amounts of rainfall. A regional PMP for the EAA Reservoir A-1 would more closely approximate what would normally be used for an on-line reservoir with an upstream drainage basin. Even a regional PMP (as defined by a storm centered over the Miami and North New River Canal watersheds) of 42 inches of average depth across the entire region is still estimated to have greater than a 10,000-year recurrence interval.

Considering the probability that the project site would experience Case 1 is, in all likelihood, as remote as the probability that it would experience Case 3, it may be advisable to consider conditions that have a higher probability of occurrence. Of the alternatives presented, Case 2, a 100-year rain event combined with a Category 5 hurricane with sustained winds of 122 mph appears to be an event that could reasonably be expected to occur within the lifetime of the EAA Reservoir A-1 and would still maintain a level of conservatism on this EAA Reservoir A-1 Project. The characteristics are about the same as a 100-year rainfall event combined with a 500-year wind. This case results in a more conservative embankment height than either Cases 1a or 3a.

We recommend that consideration be given to relaxing the wave run-up design criteria to allow the use of Case 2 to establish the embankment height. Embankment cost and savings associated with reducing the design storm to Case 2 is given in Table 21.2-3 below and show savings from the base costs given in Table 21.1-2 above.

Table 21.2-3 Redesign for Case 2 Storm: 100-year Rain, Category 5 Wind

Component	Construction Cost	Contingency at 30 percent	Total Cost
Current Embankment (Excavation, Processing, & Placement)	$ 252,600,000	$ 75,900,000	$ 328,500,000
Revised Embankment (Excavation, Processing, & Placement)	$ 233,320,000	$ 70,130,000	$ 303,450,000
		Net Savings	**$ 25,050,000**

21.2.3 Revise Operation Plan to Limit Fill Depth during Probable Hurricane Periods

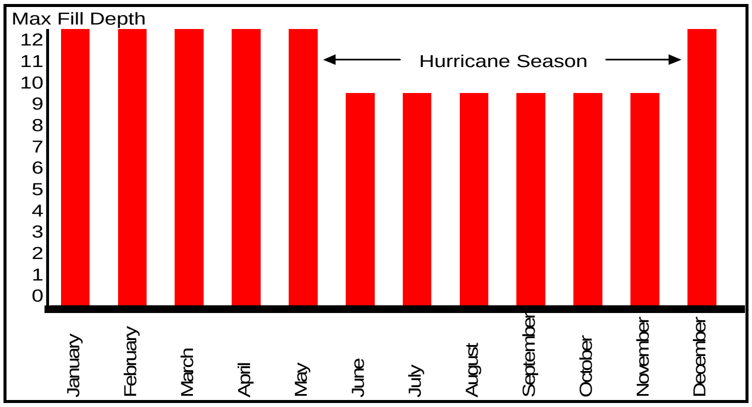
All cases considered for the wave run-up analysis are based on events that have a very small percentage of recurrence. In general, each of these cases would be associated with a major hurricane or tropical storm event. The largest of these is likely to occur during the summer and early autumn months when ocean temperatures are the highest. The potential for a Case 1 or Case 3 event is remote during the hurricane season; the potential for a maximum design event to occur outside of the hurricane season is an even more remote possibility. Therefore, an operations plan that limits the normal maximum operating pool to a stage less than 12 feet during the hurricane season would allow a corresponding reduction in embankment height.

This concept is illustrated in Figure 21.2-2. For illustration purposes, the maximum water depth during the hurricane season in this example is limited to 9 feet. During the months of June through October, the maximum operating pool level would be restricted. To ensure against over filling, the overflow spillway could be equipped with adjustable weirs that would be positioned to overflow at 9 feet of water depth during the critical months. Following the storm season, the adjustable weirs would be positioned at 12 feet of water depth to allow the remainder of the storage volume to be used.

While this option has the advantage that it would allow a cost reduction of roughly $12.5 million for every foot of embankment height reduction, it would pose an operational challenge. Operations personnel would need to follow strict guidelines in maintaining a reduced water level throughout the hurricane season and would need to be prepared to reduce EAA Reservoir A-1 water levels should a hurricane threaten during the remainder of the year. The amount of water that would need to be preemptively discharged from the EAA Reservoir A-1 under out-of-season hurricane events could be predetermined based on the anticipated precipitation and wind speeds so that operators could react accordingly.

A major disadvantage of this option is that, because the active hurricane season parallels the most abundant rainfall period, the SFWMD may not be able to capture and store precipitation to its fullest extent. Additionally, in any given year, sufficient precipitation may not be available after the hurricane season ends to allow the SFWMD to "top off" the EAA Reservoir A-1 at 12 foot depth. The depth restriction during the wet season would also compromise the flood protection capability of the system at the time when most needed.

Figure 21.2-2 Operations Plan Modification



21.2.4 Reconfigure the Embankment Perimeter

Reconfiguration of the EAA Reservoir A-1 perimeter was reviewed as another potential cost savings measure. Seven optional reservoir shapes were selected for cost analysis:

- Option A – based on reduction of potentially inefficient volume in northern half of the EAA Reservoir A-1
- Option B – based on maintenance of the eastern boundary
- Option C – based on reduction of inefficient volume in northern trapezoidal and southeastern triangular areas
- Option D – based on reduction of inefficient volume in southeastern triangular areas
- Option E – based on most efficient shape (circular)
- Options F & G – based on squaring off Option E area to create some additional volume

The areas associated with reservoir volume are shown in Figure 21.2-3 as the colored regions.

Since the current configuration as recommended in the draft BODR maximizes the available land and results in a storage volume of 190,000 acre-feet, each reconfigured reservoir shape would result in a total storage of less than 190,000 acre-feet if the 12 foot depth is maintained. Therefore, the embankment height and water depth needs to be increased for each optional reservoir shape to maintain 190,000 acre-feet of storage. This affects not only the embankment cost, but also the cost of the northeast pump station as the pumps will then be required to pump to a greater head condition. The costs for each option are summarized in Table 21.2-4.

Figure 21.2-3 EAA Reservoir A-1 Reconfiguration Options

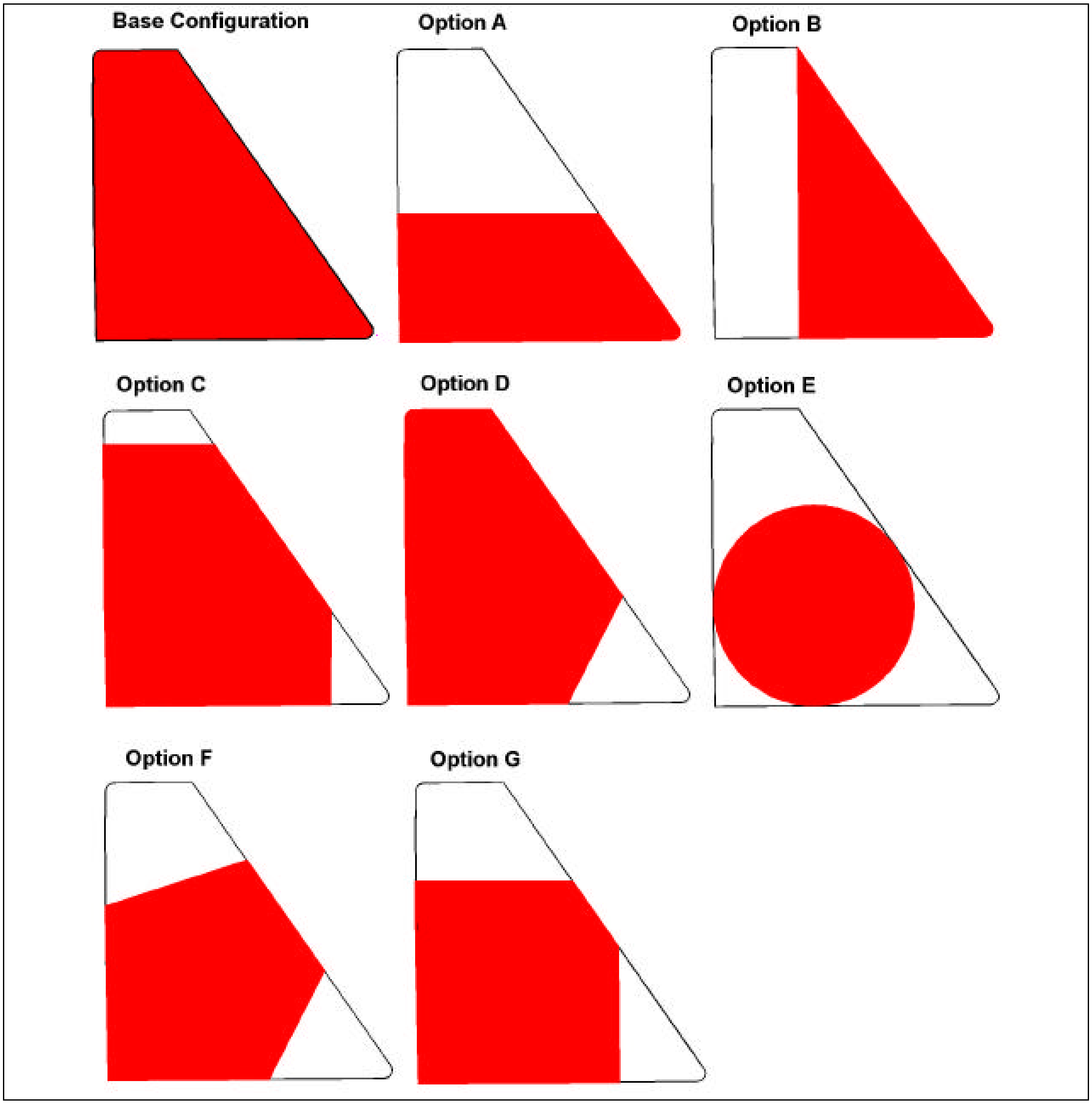


Table 21.2-4 Reconfiguration Options (Total Cost - Direct, Indirect, & 30% Contingency)

Component	Option A	Option A	Option B	Option B
Water Depth (ft) =	12	21.2	12	22.5
Storage Volume (ac-ft) =	107,700	190,000	101,520	190,000
Embankment (Excavation, Processing, & Placement)	$ 220,879,000	$ 402,881,000	$ 299,724,000	$ 417,567,000
Seepage (Canal & Cut-off)	$ 44,109,000	$ 40,677,000	$ 80,556,000	$ 74,290,000
Northeast Pump Station	$ 70,443,000	$ 120,823,000	$ 70,443,000	$ 128,232,000
Total	$ 335,431,000	$ 564,381,000	$ 450,723,000	$ 620,089,000
Net Savings =	**$ 134,240,000**	**$ (94,710,000)**	**$ 18,948,000**	**$(150,418,000)**
Volumetric Cost =	**$ 3,114**	**$ 2,970**	**$ 4,440**	**$ 3,264**
Component	Option C	Option C	Option D	Option D
Water Depth (ft) =	12	13.5	12	13.6
Storage Volume (ac-ft) =	169,128	190,000	167,250	190,000
Embankment (Excavation, Processing, & Placement)	$ 285,292,000	$ 315,617,000	$ 297,457,000	$ 316,754,000
Seepage (Canal & Cut-off)	$ 66,627,000	$ 61,444,000	$ 74,056,000	$ 68,295,000
Northeast Pump Station	$ 70,443,000	$ 76,939,000	$ 70,443,000	$ 77,509,000
Total	$ 422,362,000	$ 454,000,000	$ 441,956,000	$ 462,558,000
Net Savings =	**$ 47,309,000**	**$ 15,671,000**	**$ 27,715,000**	**$ 7,113,000**
Volumetric Cost =	**$ 2,497**	**$ 2,389**	**$ 2,642**	**$ 2,435**
Component	Option E	Option E	Option F	Option F
Water Depth (ft) =	12	21.3	12	18.1
Storage Volume (ac-ft) =	107,350	190,000	125,856	190,000
Embankment (Excavation, Processing, & Placement)	$ 252,137,000	$ 404,011,000	$ 235,320,000	$ 367,807,000
Seepage (Canal & Cut-off)	$ 76,900,000	$ 70,918,000	$ 55,078,000	$ 50,793,000
Northeast Pump Station	$ 70,443,000	$ 121,393,000	$ 70,443,000	$ 103,156,000
Total	$ 399,480,000	$ 596,322,000	$ 360,841,000	$ 521,756,000
Net Savings =	**$ 70,191,000**	**$(126,651,000)**	**$ 108,830,000**	**$ (52,085,000)**
Volumetric Cost =	**$ 3,721**	**$ 3,139**	**$ 2,867**	**$ 2,746**
Component	Option G	Option G		
Water Depth (ft) =	12	17.6		
Storage Volume (ac-ft) =	129,876	190,000		
Embankment (Excavation, Processing, & Placement)	$ 237,974,000	$ 362,143,000		
Seepage (Canal & Cut-off)	$ 53,743,000	$ 49,562,000		
Northeast Pump Station	$ 70,443,000	$ 100,306,000		
Total	$ 362,160,000	$ 512,011,000		
Net Savings =	**$ 107,511,000**	**$ (42,340,000)**		
Volumetric Cost =	**$ 2,789**	**$ 2,695**		

Cost components for embankment, seepage control components (canal and cutoff wall), and the northeast pump station are provided in Table 21.2-4, in addition to the cost savings or increase compared to the costs for the current configuration.. Additionally, a volumetric cost ($/acre-feet of storage) is provided for each option. The volumetric cost for the embankment, seepage control, and northeast pump station configuration included in the BODR was $2,443/acre-feet for these same components. It is noted that although there is substantial cost savings for these configurations if the 12 foot depth is maintained, the overall storage drops significantly below the target 190,000 acre-feet. In order to maintain the storage target, so there is no negative impact on the EAA Reservoir A-1's ability to meet environmental or agricultural deliveries, costs for most options actually increase.

Two options do result in cost savings: Option C and Option D. Both options reduce the area of the inefficient triangular space in the south east corner of the site. The volume reduction for this area is proportionately less than the reduction in perimeter, resulting in a higher volume-to-perimeter ratio than the proposed configuration. Unfortunately, this is also the location of existing G-370 pump station. Although G-370 pump station could be made to function with this configuration in an unmodified condition, plans to modify G-370 pump station to pump to a 12-foot water depth in the future would be rendered useless. The cost of infrastructure improvements to allow G-370 modifications in the future would far exceed any savings to the embankment cost at this time. Therefore, neither of these are viable options.

It should be noted that the options include a circular configuration. Although a circular configuration should be the most efficient shape, the cost analysis shows that it is not the most cost effective. This is because the unit cost of the embankment adjacent to the supply canal is less than the cost of the remainder of the embankment because it takes advantage of the existing supply canal levee and has reduced seepage provisions. The circular configuration does not take advantage of incorporating the STA-3/4 Supply Canal Levee, and consequently results in higher per foot costs. Due to the length of the lower cost embankment adjacent to the Supply Canal relative to the remaining embankment length, the current configuration is actually one of the most cost effective shape of those evaluated. Consequently, reconfiguring the EAA Reservoir A-1 is not recommended as a cost savings measure.

21.2.5 Limit EAA Reservoir A-1 Operational Depth at 8 foot During Phase 1

Limiting the maximum operating depth of the EAA Reservoir A-1 to 8-ft until the EAA Reservoir A-2 is completed in 2015 as currently projected could result in some cost savings. The impact of this operational decision would be limited to the embankment section adjacent to the EAA Reservoir A-2 area. This section of the embankment can be lowered by 4 feet to accommodate the 8 foot Phase 1 depth, wave run-up, and wind setup. The remainder of the EAA Reservoir A-1 embankment would be constructed at the height necessary for the full 12 feet of EAA Reservoir A-1 depth to accommodate full operating depth once EAA Reservoir A-2 is constructed. The breakdown of costs and savings associated with lowering the approximately 15,000 ft of embankment adjacent to EAA Reservoir A-2 is presented in Table 21.2-5. We would not recommend a similar height reduction in the remainder of the embankment. The cost to increase the embankment height when EAA Reservoir A-2 is constructed would be greater than any savings that could be experienced at this time.

Table 21.2-5 Phase 1 Depth of 8 feet

Component	Construction Cost	Contingency at 30 percent	Total Cost
Current Embankment (Excavation, Processing, & Placement)	$ 252,600,000	$ 75,900,000	$ 328,500,000
Revised Embankment (Excavation, Processing, & Placement)	$ 248,500,000	$ 74,700,000	$ 323,200,000
		Net Savings =	**$ 5,300,000**

The negative impact of this operational decision is reflected in the EAA Reservoir A-1's ability to make deliveries to the environmental and agricultural areas. Applying the water balance model (WBM) for the period of simulation, an 8 foot deep reservoir was compared to a 12 foot deep EAA Reservoir A-1. This exercise incorporated, a 1,500 cfs northeast pump station pumping to either 8 or 12 foot of EAA Reservoir A-1 depth as appropriate and existing pump stations G-370 and G-372 unmodified pumping to 8 foot of EAA Reservoir A-1 depth at 2,350 cfs and 3,130 cfs, respectively. The results of that analysis are presented in Table 21.2-6.

Table 21.2-6 Deliveries Met at Various Depths

	2010 Flows, 2010 Environmental & 2010 Agricultural Deliveries		**2010 Flows, 2015 Environmental & 2010 Agricultural Deliveries**	
Depth	Environmental Deliveries Met	Irrigation Deliveries Met	Environmental Deliveries Met	Irrigation Deliveries Met
8	99.6 percent	85.7 percent	42.3 percent	28.3 percent
12	99.7 percent	91.3 percent	44.2 percent	31.4 percent

21.2.6 Addition of an Uncontrolled Spillway

At the request of SFWMD representatives, the addition of an uncontrolled crest spillway was evaluated to determine whether sufficient capacity could be removed from the EAA Reservoir A-1 during storm events to allow a reduction in embankment height. This analysis is summarized in Section 6. The analysis indicated that the costs associated with the addition of an uncontrolled spillway would exceed the potential savings resulting from a corresponding reduction in embankment height. Consequently, addition of an uncontrolled spillway to reduce EAA Reservoir A-1 embankment height is not recommended.

21.2.7 Reduction in Pump Station Capacity

The 3,600 cfs capacity recommended for the northeast pump station in Section 6.5 is based on optimizing priority removals. As discussed in that section, the minimum recommended size for that station to optimize deliveries is 1,500 cfs. One option to reduce cost would be to construct a reduced capacity pump station to meet delivery optimization. The disadvantage of this is that SFWMD would have a greatly reduced capacity for run-off removal and flood protection, to meet priority removals resulting in more pump-backs to Lake Okeechobee, and to limit direct, high volume discharge to STA-3/4.

The smaller facility would save about $33.7 million in pump station cost. In addition, the cost associated with increased canal capacity could be avoided saving another $36.7 million (not currently included in this EAA Reservoir A-1 Project's opinion of probable cost).

21.2.8 Reduce Contingency

The contingency in the July 29, 2005 submittal was presented as 30 percent based on DCM guidelines. Based on the current level of detail for the EAA Reservoir A-1 embankment reflected by the opinion of probable cost and previous cost opinions received for similar pump stations, we recommend a reduction in the cost contingency to 20 percent for both the embankment and the pump station. We recommend that the contingency for the control structures and the U.S. 27 bridge remain at 30 percent. A summary of the reduction of the various contingencies is given in Table 21.2-7.

Table 21.2-7 Reduction of Contingency to 20 percent

Structure	Contingency at 30 percent	Contingency at 20 percent	Cost Reduction
Embankment Contingency	$ 75,926,000	$ 50,515,000	$ 25,411,000
Seepage Control Contingency	$ 17,387,000	$ 11,588,000	$ 5,799,000
Northeast Pump Station Contingency	$ 21,120,000	$ 14,080,000	$ 7,040,000
Control Structures Contingency	$ 4,400,000	n/a	n/a
US 27 Bridge Contingency	$ 1,600,000	n/a	n/a
Total Contingency/Savings	$ 120,433,000	$ 80,163,000	$ 38,250,000

At this time, we would not recommend further reduction in the contingency. In light of current events in the gulf region following the 2005 hurricane season, fuel prices remain in a volatile state. These circumstances have the potential of increasing the demand for construction laborers in the South, which could affect wage rates for the EAA Reservoir A-1 Project. Additionally, market volatility of prices for cement (as would be needed for the roller compacted concrete (RCC) embankment) and other commodities could also impact the total project cost.

The cost opinion included in Section 23 has been adjusted to reflect a reduction in the contingency as recommended herein.

21.2.9 Combinations

Finally, each of the cost reducing measures described herein are not mutually exclusive and could be incorporated in combination to result in greater cost savings. If all the potential costs saving measures were implemented, savings could exceed $100 million. Potential combinations are listed in Table 21.2-8.

Table 21.2-8 Combinations

Measure No.							Individual Savings
1. Incorporate Parapet							$ 10,331,000
2. Change Design Storm							$ 25,052,000
3. Seasonal Operating Depth of 9-ft							$ 37,520,000
5. Lower Phase 1 Depth to 8-ft							$ 5,299,000
8. Reduce Contingency							$ 38,250,000
	1	2	3	5	8	Total Height Reduction (*)	Total Net Savings
A	Y	Y			Y	3.5	$ 73,633,000
B	Y		Y		Y	4.5	$ 86,101,000
C	Y			Y	Y	1.5 (5.5)	$ 53,880,000
D	Y	Y	Y		Y	6.5	$ 111,153,000
E	Y	Y		Y	Y	3.5 (7.5)	$ 78,932,000
F	Y		Y	Y	Y	4.5 (8.5)	$ 91,400,000
G	Y	Y	Y	Y	Y	6.5 (10.5)	$ 116,452,000
H		Y	Y		Y	5	$ 100,822,000
I		Y		Y	Y	2 (6)	$ 68,601,000
J		Y	Y	Y	Y	5 (9)	$ 106,121,000
K			Y	Y	Y	3 (7)	$ 81,069,000
(*) value in parentheses applies only to embankment along the future A-2 area							

21.3 REFERENCES

DeMaria, Mark and John Kaplan, National Oceanic & Atmospheric Administration, "The Inland Wind Model and the Maximum Envelope of Winds (MEOW), http://www.nhc.noaa.gov/aboutmeow.shtml.

SECTION 22

CONSTRUCTION COORDINATION

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22. CONSTRUCTION COORDINATION

22.1 INTRODUCTION

The EAA Reservoir A-1 project will include the construction of about 22 miles of embankment around the perimeter of an approximate 16,000 acre site. The southern boundary and the southern portion of the western boundary is bordered by the STA-3/4 Supply Canal. The eastern boundary abuts the right-of-way for U.S. 27 and the remainder is adjacent to agricultural property. The agricultural property on the western boundary is part of the future EAA Reservoir A-2. A buffer is planned between the EAA Reservoir A-1 embankment and the active agricultural areas on the west and north sides.

22.1.1 Perimeter Embankment

An embankment is recommended as the most cost effective solution for the EAA Reservoir A-1 containment. The embankment will be constructed from materials excavated from adjacent canals and from materials manufactured from one or more on site borrow areas. Where not adjacent to the existing STA-3/4 Supply Canal, a slurry cutoff wall will be constructed to reduce seepage and protect the embankment. An exterior seepage canal will provide embankment materials and will collect seepage for return to the EAA Reservoir A-1. The remainder of the embankment material, including rock for processing drainage and filter materials, will be excavated from an internal canal parallel to the embankment or extended borrow areas. Roller compacted concrete as facing and in steps will be used for slope protection.

22.1.2 Pump Stations

Three pump stations will supply water to the EAA Reservoir A-1. Existing G-370 and G-372 pump stations were constructed to supply water to STA-3/4, and will also supply the EAA Reservoir A-1 in the future. These existing stations will be utilized without modification to deliver water to the EAA Reservoir A-1 when the reservoir water level is at or below a level where the existing pumps can operate efficiently. A new northeast pump station will be constructed near the northeast corner of EAA Reservoir A-1 and will deliver water from the NNRC into the EAA Reservoir A-1. The size of the new pump station will depend on whether the SFWMD decides to eliminate pump backs to Lake Okeechobee. A new Supply Canal will connect the NNRC to the pump station. The northeast pump station will be equipped with pumps to return seepage to the EAA Reservoir A-1. Seepage pumps located in the G-370 and G-372 pump stations are expected to remain in service to supplement storm water runoff and seepage pumping capability.

22.1.3 Control Structures

Three gate structures will be constructed in the embankment to control the flow of water into and out of the EAA Reservoir A-1. Two structures will be located in the embankment common to the STA-3/4 Supply Canal and EAA Reservoir A-1. These structures will serve the dual purpose of filling the EAA Reservoir A-1 when water is available and the EAA Reservoir A-1 level allows, and also for discharging water to the Supply Canal for environmental purposes. These structures will also be available for emergency discharges to the STA-3/4 when the EAA Reservoir A-1 is above the normal operating pool.

A third structure will be constructed near the northeast pump station to release water from the EAA Reservoir A-1 into the NNRC. The primary purpose for releases to the NNRC will be for meeting agricultural deliveries, but the structure may also be used for emergency releases if the EAA Reservoir A-1 reaches critical water levels, or if emergency draw-downs are required. If the water level is too low to allow gravity discharge directly to the Supply Canal, water will be released at this location from where it can be pumped by G-370 pump station for environmental deliveries. An orifice type spillway will also be incorporated into the structure.

22.2 PERMITS

Refer to Section 4 for a listing of required permits and approvals.

22.3 SCHEDULE

In general, construction of the EAA Reservoir A-1 Project will commence in the spring of 2006 and be completed in the fall of 2009. As design progresses, decisions will be finalized with regard to the specific break down and scheduling of construction contracts. It is expected that four major construction packages will be implemented.

- Reservoir Embankment and Control Structures Construction
- Pump Station Construction
- Pumping Equipment Procurement
- U.S. 27 Bridge Construction

These packages could be broken down into smaller packages to spread the work and to facilitate earlier construction. Specific targets for advertisement, award, completion, and startup for each packages will be established during detailed design.

22.4 OTHER PROJECTS AFFECTING CONSTRUCTION

Coordination will be required with the following construction contracts, which may be underway concurrently with the EAA Reservoir A-1 contracts:

- U.S. 27 expansion (FDOT)
- Compartment B Construction
- NNRC enlargement

No other projects are known to be under construction in the immediate vicinity of the EAA Reservoir A-1 Project; however, close coordination between the major construction contracts for this Project will be required. Because of the general increase in construction associated with the Acceler8 program, some shortage of local labor, materials, and equipment can be expected.

22.5 CONSTRUCTION SEQUENCING AND STAGING

22.5.1 General

The EAA Reservoir A-1 Project will involve several contractors working simultaneously to complete the work within the desired schedule. The specific sequencing of the components for each construction contract will be developed by the construction contractor using constraints that will be specified in the construction documents. The major constraints are discussed in subsequent paragraphs.

22.5.2 Perimeter Embankment

Most of the materials to be used in the embankment will be excavated from the external seepage canal and an interior borrow canal or borrow area. During the construction of embankments for two Test Cells in early 2005, it was determined that excavated materials are difficult to adequately dewater for direct placement in the embankment. Therefore, it may be prudent to excavate and stockpile materials for two to three months, or more, in advance of embankment construction. Further testing of test cell embankment materials is required to determine whether or not this would be beneficial. Pre-excavation and embankment construction could be performed under separate contracts or under one contract with appropriate sequencing time between excavation and embankment fill placement. The embankment will be about 22 miles in length so even if the excavation and placement are included in a single contract, there will be ample opportunity for a single contractor to excavate embankment materials well in advance of the placement.

For purposes of this discussion, it will be assumed that the excavation and placement is completed under a single contract. Caprock will be blasted in the canals and could be placed directly in the rockfill section of the embankment except for the rock to be used for rock processing and slope protection. Rock to be processed for RCC, required for erosion protection, can be excavated from the interior borrow canal, sorted as necessary, and stockpiled for later use. The remaining material from the borrow canals will be used in the random fill section of the embankment. This material will be stockpiled and allowed to drain for an extended period before being placed in the embankment.

Along the STA-3/4 Supply Canal, the EAA Reservoir A-1 embankment will abut the northern Supply Canal embankment and will extend across the existing seepage canal for the Supply Canal. In order for the seepage canal to remain in service during construction, the embankment construction will commence from the southwest corner of the EAA Reservoir A-1 toward the pump stations, thereby allowing the seepage canal to remain in use. Maintaining the seepage canal in use will assist in dewatering the embankment foundation area during construction.

Blasting will be necessary for the seepage canal excavation. Close coordination with the FDOT will be required along the eastern boundary to protect motorists from debris. It is anticipated that traffic stoppages will be required when blasting is being conducted.

Two or more quarry operations will be developed within the EAA Reservoir A-1 site for producing filter and drain materials for the embankment. The locations will be determined during design when geotechnical investigations have been completed and areas with suitable caprock thickness and quality have been identified.

22.5.3 Pump Stations

The new northeast pump station will be located immediately adjacent to U.S. 27. It is expected that the pump station will be constructed under a separate contract from the embankment. Coordination between the two contracts will be necessary for the portion of the embankment where the pump station will be constructed. It is anticipated that the embankment contractor will leave a gap in the embankment for the pump station construction and subsequently complete the tie in to the pump station after it has been completed.

It will be critical to maintain continuous operations of STA-3/4 during construction. The preferred alternative for the existing G-370 and G-372 pump stations would be to utilize them for partial EAA Reservoir A-1 filling without modification. For that alternative, disruption to STA-3/4 operations would be limited to construction of gate structures in the common embankment between the Supply Canal and the EAA Reservoir A-1. If the existing pump stations are modified to pump to the full EAA Reservoir A-1 elevation, modifications to the pump stations within the Supply Canal will be required. For that scenario, work will be sequenced such that only one pump bay is out of service at any time. Short periods of time will be scheduled for taking the Supply Canal out of service on one side or the other of gate structure G-383 for work on the pump stations, or for installation of additional gate structures within the Supply Canal.

22.5.4 Control Structures

It is expected that control structures in the embankment will be constructed under a separate contract. Structures in the common embankment between the EAA Reservoir A-1 and Supply Canal will require cofferdam construction to allow the structures to be constructed without taking the Supply Canal out of service. Temporary access around new structure areas will be required for SFWMD maintenance operations during construction of each structure. It will be necessary to maintain access around the southeast and southwest gate structures for fuel deliveries to the G-372 pump station. The gates near the northeast pump station will be constructed as part of the embankment contract.

22.5.5 Agricultural Operations

Several agricultural canals traverse the EAA Reservoir A-1 site and supply water to farming operations within the EAA Reservoir A-1 footprint. Areas to the west of the EAA Reservoir A-1 can probably be supplied from the Miami Canal. If insufficient existing pump capacity is available, the contractor will be required to provide temporary pumping from the Miami Canal or the new seepage canal for irrigation and drainage within EAA Reservoir A-1. It is currently anticipated that these canals must remain in service during construction of the EAA Reservoir A-1. As such, the embankment that crosses these canals will be constructed near the end of the construction period and will be coordinated to minimize disruption of agricultural deliveries during the growing season. Once the canals have been dammed by the embankment, the Contractor will be required to maintain temporary irrigation and drainage pumping for the remainder of the growing/harvest season for that year's crop. At that time the Contractor will also demolish the existing agricultural pump stations along U.S. 27 and complete the embankment through those areas.

22.5.6 Staging

A staging area will be provided at site for the new northeast pump station. For the embankment construction it is expected that accesses off of U.S. 27 will be utilized and staging areas will be developed inside of the EAA Reservoir A-1. The number of staging areas will depend on the number of contracts used for the earthwork. Locations and size will be established during subsequent design phases. Contractors may establish minor staging areas around the perimeter of the embankment to accommodate construction.

Secondary staging locations will be established at the quarry operations. Space is available for staging at the existing pump stations if an alternative is selected which requires their modification. It is expected that a staging area near the G-370 pump station will be established for construction of the control structures between the EAA Reservoir A-1 and the STA-3/4 Supply Canal.

22.6 DISPOSAL

The agricultural pump structures located along U.S. 27 will be demolished by the Contractor for the embankment construction and the materials will be disposed of by the Contractor. The SFWMD may determine that certain mechanical equipment should be delivered to a location of their choice, as set out in the contract documents.

The interior of the EAA Reservoir A-1 site contains little infrastructure. All of the buildings have been removed from the old mill site and only concrete slabs remain. The concrete slabs will remain after construction. A power pole line which once served the old mill site, and currently serves agricultural pumps, will be demolished and disposed of by the Contractor.

The only other known structures on the EAA Reservoir A-1 site are at-grade structures such as canal water control features and culverts. These will have no negative impact on the completed EAA Reservoir A-1, and will therefore remain.

22.7 QUALITY ASSURANCE

Quality assurance requirements will be developed during design in conjunction with the Construction Manager and the requirements of the SFWMD for the Acceler8 program, and will be specified in the Contract Documents.

SECTION 23

OPINION OF PROBABLE COST

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TABLES

23. OPINION OF PROBABLE COST

23.1 INTRODUCTION

An opinion of probable cost has been developed for the Project that is considered to be consistent with the understanding of the Project requirements and knowledge of the conditions investigated to date. Development of the probable costs for the EAA Reservoir A-1 is advanced in more detail than typically developed at a basis of design phase due to the accelerated EAA Reservoir A-1 design activities including completion of the Test Cell Program. The methodology used to compile this cost opinion for the Project components included estimating quantities and defining a construction procedure for constructing the components. The cross-section for the embankment and the construction process were developed from experience gained from the Test Cell construction program. Pump station, control structure, bridge, and canal probable costs are based on past development costs experienced by the SFWMD and Black & Veatch for recent construction of similar types and sizes of facilities.

23.2 DIRECT COSTS

23.2.1 Embankment and Canals

The opinion of probable cost for the embankment was developed for constructing the preferred embankment cross-section defined in Section 8. Material quantities for the embankment sections were determined and a construction plan for obtaining the required materials on site and placement in the embankment was developed. The construction plan included the process for mobilization; excavation and dewatering; hauling and stockpiling; foundation cutoff installation; foundation preparation and treatment; embankment materials placement and compaction; material processing; and demobilization.

This plan was used to define contractor direct costs for completing the planned construction. The direct costs have been defined for major cost components of labor, material, and equipment required to complete the construction. Subcontract and other costs have been included for specialty contractor work. A construction duration of 30 months was considered for selecting appropriate construction crew size and equipment requirements. The opinion of probable cost was based on two, 10-hour shifts per day production for six days per week. Using the estimated quantities of work to complete the construction, pricing was applied using RS MEANS 2005 for labor production man-hours per unit of work to define labor, material, and equipment costs. Labor costs were based on RS MEANS 2005 base wage rates plus fringe benefit package adjusted to Tampa, Florida. Monthly equipment rental rates and hourly operating costs were used in the development of the opinion of probable costs.

Subcontract costs included in the cost opinion are based on verbal communication with specialty contractors.

All pricing was extended for the work quantities to compile the contractor's direct cost. Mobilization/demobilization were added and are based on estimated transportation costs.

The summary of extended pricing is presented in a standard spreadsheet format identifying the cost categories and division of work activities considered for the embankment construction. The

Direct Total Cost is shown on the first page of the cost summary by tasks and as Construction Subtotal (Direct Costs) on the detailed opinion of probable cost work breakdown.

23.2.1.1 Northeast Pump Station

The opinion of probable cost for the northeast pump station was developed for the preferred alternative. The pump station will be located near the northeast corner of the EAA Reservoir A-1 and will be comprised of six equal sized pumps with a total capacity of 3,600 cfs. The probable cost includes the pump station, connector canal, and site work.

23.2.1.1 Control Structures

An opinion of probable cost has been included for three control structures. The southwest gate structure is a combination inlet/outlet structure located in the embankment along the STA-3/4 Supply Canal. It is comprised of eight box culverts under the embankment, each equipped with a cable drawn stainless steel roller gate. The southeast gate structure is similar to the southwest gate structure except that it is comprised of five box conduits.

The northeast gate structure is located near the northeast pump station and serves as a discharge structure for the EAA Reservoir A-1. It will have five box conduits and will discharge to the connector canal for the northeast pump station. Each conduit will be equipped with a cable drawn stainless steel roller gate. Incorporated into this structure will be an uncontrolled, fixed weir-type, trough spillway that will connect to an ungated conduit parallel to the five box conduits.

23.2.2 US 27 Bridges

Two bridges are required over the connector canal for the northeast pump station, one for each direction of traffic on US 27.

23.3 INDIRECT COSTS

Indirect costs are added to the direct costs to complete the total cost for the embankment construction. The contingency of 30 percent was established by Accele8 guideline Design Criteria Memorandum DCM-7, for the BODR phase cost opinions. Due to the advanced design of the embankment and understanding of foundation conditions as a result of the Test Cell Program and geotechnical investigations, it is appropriate to reduce the Project contingency for the embankment to 20 percent. Likewise, the level of detail provided by SFWMD's standard details for large pump stations also justifies the use of a 20 percent contingency for the northeast pump station. All other components of the Project will use a contingency of 30 percent in accordance with DCM-7.

The indirect costs included in the opinion of probable cost are defined as follows:

Sales Tax	6 percent of purchased materials + equipment rental
General Requirements	9 percent of construction costs
Overhead and Profit	17 percent of construction cost + general requirements
Bonds and Insurance + overhead and profit	3.5 percent of construction cost + general requirements + sales tax

Project Reserve	5 percent of construction cost
Contingency	30 percent or 20 percent (see discussion above) of construction cost + general requirements + sales tax + overhead and profit + bonds and insurance + escalation + project reserves

23.4 BODR OPINION OF PROBABLE COST

The Opinion of Probable Cost for the BODR phase of the EAA Reservoir A-1 Project is provided in Table 23.4-1. The detailed Opinion of Probable Costs for the embankment is presented in Table 8.10-1 in Section 8 of this BODR. Detailed cost opinions for the Northeast Pump Station, control structures, and the US 27 bridge are included at the end of this section.

Table 23.4-1 Summary Opinions of Probable Cost

Project Component	Description	Direct Cost (millions)	Indirect Costs (millions)			Total Cost (millions)
			Construction Indirects*	Project Reserve	Contingency	
Embankment and canals	Excavation Embankment Slope Protection Cutoff Seepage Canal Rock Processing Imported Materials	$ 206.6	$ 89.0	$ 14.8	$ 62.1	**$ 372.5**
Northeast Pump Station	Pump Station Structures Pumps (6) Mechanical Equipment Electrical Equipment Connection Canal Site Work	$ 50.3	$ 16.8	$ 3.3	$ 14.1	**$ 84.5**
Control Structures	Southwest Gate Structure Southeast Gate Structure Northeast Gate Structure and Spillway	$ 10.4	$ 3.5	$ 0.7	$ 4.4	**$ 19.0**
US 27 Bridge	Bridges (2)	$ 3.8	$ 1.2	$ 0.3	$ 1.6	**$ 6.9**
	Totals	**$ 271.1**	**$ 110.5**	**$ 19.1**	**$ 82.2**	**$ 482.9**

* Construction indirect costs include sales tax, general requirements, overhead and profit, and bonds and insurance.

23.5 OPINION OF PROBABLE COST ATTACHMENTS

- Embankment
- Northeast Pump Station
- Control Structures
- US 27 Bridge

Table 23.5-1 Opinion of Probable Cost – Embankment and Canals

ITEM No.	DESCRIPTION	Quantity	Unit	Unit Cost	Man-Hours	Labor Cost	Material Cost	Equipment Cost	Subcontract Cost	Other Cost	Direct Total Cost	Indirects Mark-Ups	Indirect Total Cost	Total
1	**Strip Peat**													
	Materials & Methods				55,946	955,274	0	5,976,601	0	0	6,931,875	0.8032	5,567,445	12,499,320
	Subtotal				55,946	$955,274	$0	$5,976,601	$0	$0	$6,931,875		$5,567,445	$12,499,320
2	**Seepage Collection Canal Construction (Cap Rock Removal)**													
	Materials & Methods				183,658	3,015,790	2,297,434	12,164,529	0	0	17,477,752	0.8032	14,037,534	31,515,287
	Subtotal				183,658	$3,015,790	$2,297,434	$12,164,529	$0	$0	$17,477,752		$14,037,534	$31,515,287
3	**Seepage Collection Canal Construction (Excavate Silty Sand, Limestone, Etc. Soils)**													
	Materials & Methods				33,455	482,299	0	3,125,807	0	0	3,608,106	0.8032	2,897,908	6,506,014
	Subtotal				33,455	$482,299	$0	$3,125,807	$0	$0	$3,608,106		$2,897,908	$6,506,014
4	**Embankment Construction (Production Blast Cap Rock & Excavate Silty Sand, etc.)**													
	Materials & Methods				609,596	9,536,983	5,683,347	39,056,936	0	0	54,277,266	0.8032	43,593,649	97,870,915
	Subtotal				609,596	$9,536,983	$5,683,347	$39,056,936	$0	$0	$54,277,266		$43,593,649	$97,870,915
5	**Embankment Construction (Cap Rock Crushing)**													
	Materials & Methods				103,477	1,707,431	0	1,790,785	0	8,220,329	11,718,545	0.8032	9,411,936	21,130,481
	Subtotal				103,477	$1,707,431	$0	$1,790,785	$0	$8,220,329	$11,718,545		$9,411,936	$21,130,481
6	**Embankment Construction (Surface Preparation / Cut-off Wall)**													
	Materials & Methods				70,505	1,073,838	5,925,920	1,986,478	5,805,122	0	14,791,358	0.8032	11,879,914	26,671,273
	Subtotal				70,505	$1,073,838	$5,925,920	$1,986,478	$5,805,122	$0	$14,791,358		$11,879,914	$26,671,273
7	**Embankment Construction (Sand filters and Drains)**													
	Materials & Methods				40,407	647,012	0	1,375,929	1,316,717	0	3,339,657	0.8032	2,682,299	6,021,956
	Subtotal				40,407	$647,012	$0	$1,375,929	$1,316,717	$0	$3,339,657		$2,682,299	$6,021,956
8	**Embankment Construction (Rock Fill)**													
	Materials & Methods				439,113	6,822,119	0	29,421,122	0	927,217	37,170,457	0.8032	29,854,043	67,024,501
	Subtotal				439,113	$6,822,119	$0	$29,421,122	$0	$927,217	$37,170,457		$29,854,043	$67,024,501
10	**Embankment Construction (Topsoil and Seeding)**													
	Materials & Methods				25,738	392,537	144,172	1,181,367	0	0	1,718,075	0.8032	1,379,900	3,097,975
	Subtotal				25,738	$392,537	$144,172	$1,181,367	$0	$0	$1,718,075		$1,379,900	$3,097,975
11	**Embankment Construction (Cut-off Wall Cap, Concrete Face, & Parapet)**													
	Materials & Methods				415,861	6,103,947	31,966,704	17,309,216	0	0	55,379,867	0.8032	44,479,220	99,859,087
	Subtotal				415,861	$6,103,947	$31,966,704	$17,309,216	$0	$0	$55,379,867		$44,479,220	$99,859,087
12	**Equipment Mobilization Or Demobilization**													
	Materials & Methods				929	11,560	0	52,728	0	103,249	167,538	0.8032	134,561	302,099
	Subtotal				929	$11,560	$0	$52,728	$0	$103,249	$167,538		$134,561	$302,099
	Total				1,978,684	$30,748,789	$46,017,577	$113,441,498	$7,121,839	$9,250,795	$206,580,498		$165,918,409	$372,498,908

						Labor						Material		Equipment						
CSI Div. / Sect.		DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost
1		**General Requirements**																		
		Mobilization	1	LS	2,065,805						0.00		0.00						2,065,805	2,065,805
		Supervision	1	LS	10,329,025						0.00		0.00						10,329,025	10,329,025
		Temporary construction facilities	1	LS	5,164,512						0.00		0.00						5,164,512	5,164,512
		Temporary utilities	1	LS	3,098,707						0.00		0.00						3,098,707	3,098,707
		Safety	1	LS	5,164,512						0.00		0.00						5,164,512	5,164,512
		Miscellaneous	1	LS	4,131,610						0.00		0.00						4,131,610	4,131,610
		Subtotal Mobilization									$0		$0				$0	$0	$29,954,172	$29,954,172
2		**Site Work**																		
02225		**Demolition**																		
02230		**Site Clearing**																		
SC	1	Scraper (Strip Peat Canal Area)	506,489	CY	2.06	B5b	0.012	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626
SC	1	Scraper (Strip Peat Bench Area)	1012978	CY	1.55	B5b	0.009	9,117	455.8	17.17	156,498.99	0.00	0.00	1s1	2	154.87	1,411,940.15		0	1,568,439
EM	1	Scraper (Strip Peat Embankment A	915555	CY	1.03	B5b	0.006	5,493	274.7	17.17	94,298.47	0.00	0.00	1s1	2	154.87	850,764.54		0	945,063
EM	1	Surface Prep. (Strip Peat Embankr	1373332	SY	0.35	B5	0.008	10,987	915.6	17.22	189,226.83		0.00	3j4+	2	26	289,566.31		0	478,793
EM	1 W/O	Scraper (Strip Peat Embankment A	364980	CY	1.03	B5b	0.006	2,190	109.5	17.17	37,591.48	0.00	0.00	1s1	2	154.87	339,151.84		0	376,743
EM	1 W/O	Surface Prep. (Strip Peat Embankr	547470	SY	0.35	B5	0.008	4,380	365.0	17.22	75,434.07		0.00	3j4+	2	26	115,433.76		0	190,868
EM	1	Scraper (Strip Peat Inside Bench A	1519467	CY	0.69	B5b	0.004	6,078	303.9	17.17	104,332.66	0.00	0.00	1s1	2	154.87	941,293.44		0	1,045,626
EM	1 W/O	Scraper (Strip Peat Inside Bench A	1026667	CY	0.69	B5b	0.004	4,107	205.3	17.17	70,495.04	0.00	0.00	1s1	2	154.87	636,009.08		0	706,504
EM	1	Dozer Angle Blade (Strip Peat Insi	485400	CY	1.15	B2	0.015	7,281	455	16.37	119,190	0.00	0.00	6c	1	120.02	436,945.59		0	556,135
EM	1 W/O	Dozer Angle Blade (Strip Peat Insi	15777	CY	1.15	B2	0.015	237	15	16.37	3,874	0.00	0.00	6c	1	120.02	14,202.41		0	18,077
02240		**Dewatering**																		
		Seepage Canal																		
SC	2	Pump,12" Suction (Make-up)	29,060	LF	68.94	B4	0.040	1,162	145.3	16.87	19,606.64	0.00	0.00			68.27	1,983,900.00		0	2,003,507
SC	2	12" Dia Pipe 2,500 GPM	54,701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925
		Production Blast																		
SC	4 w/o	Pump,12" Suction (Make-up)	25780	LF	68.94	B4	0.040	1,031	128.9	16.87	17,393.49	0.00	0.00			68.27	1,759,962.16		0	1,777,356
SC	4 w/o	12" Dia Pipe 2,500 GPM	91661	LF	5.48	B4	0.098	8,983	1,122.8	16.87	151,516.68	0.00	0.00			3.83	351,060.86		0	502,578
		Cut Off Wall																		
SC	2 w/o	Pump,12" Suction (Make-up)	18803	LF	68.94	B4	0.040	752	94.0	16.87	12,686.65	0.00	0.00			68.27	1,283,700.00		0	1,296,387
SC	2 w/o	12" Dia Pipe 2,500 GPM	54701	LF	5.48	B4	0.098	5,361	670.1	16.87	90,421.24	0.00	0.00			3.83	209,504.06		0	299,925
02300		**Earthwork**																		
02305		**Equipment Mobilization Or Demobilization**																		
	12	Dump Truck (26 Tons)	84	EA	346	C1	2.000	168	21.0	11.88	1,995.84	0.00	0.00	3e1	1	49.87	8,378.71		18,674	29,049
	12	Dozers (Above 150 HP)	44	EA	578	C1	2.667	117	14.7	11.88	1,394.09	0.00	0.00	3g	2	65.48	7,683.36		16,339	25,417
	12	Front Loaders	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Vibrating Roller	16	EA	578	C1	2.667	43	5.3	11.88	506.94	0.00	0.00	3g	2	65.48	2,793.95		5,942	9,242
	12	Crawler Type Drill, 4"	24	EA	322	C2	6.000	144	18.0	15.55	2,239.20	0.00	0.00	6h	4	34.56	4,977.29		520	7,736
	12	Air Compressor , 600 CFM	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0
	12	50 Ft Air Hose, 3" Dia.	24	EA	0	C2	0.000	0	0.0	15.55	0.00	0.00	0.00	6h	4	34.56	0.00		0	0
	12	Excavator, Diesel Hydraulic, Crawl	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777
	12	Crusher	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Concrete Batch Plant, Portable, 20	8	EA	3,466	C1	16.000	128	16.0	11.88	1,520.64	0.00	0.00	3g	2	65.48	8,380.80		17,823	27,724
	12	Concrete Transit Mixer Truck	24	EA	461	C1	2.667	64	8.0	11.88	760.42	0.00	0.00	3e1	1	49.87	3,192.29		7,115	11,068
	12	Grader 30,000 Lbs.	2	EA	578	C1	2.667	5	0.7	11.88	63.37	0.00	0.00	3g	2	65.48	349.24		743	1,155
	12	Scraper, Self-Propelled, 32-44 Cy	10	EA	578	C1	2.667	27	3.3	11.88	316.84	0.00	0.00	3g	2	65.48	1,746.22		3,714	5,777
	12	Truck Mtd. Crane Over 75 Ton	8	EA	1,386	C1	6.400	51	6.4	11.88	608.26	0.00	0.00	3g	2	65.48	3,352.32		7,129	11,090
	12	Attachment Concrete Bucket, 8 CY	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Rubber tired backhoe-loader, 3/4 C	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
	12	Wheeled Skid Steer, Diesel, w/ Brc	14	EA	578	C1	2.667	37	4.7	11.88	443.58	0.00	0.00	3g	2	65.48	2,444.71		5,199	8,087
	12	Hoe Rams	4	EA	578	C1	2.667	11	1.3	11.88	126.74	0.00	0.00	3g	2	65.48	698.49		1,485	2,311
	12	Wash & Screen (Sand Horiz. Blank	8	EA	578	C1	2.667	21	2.7	11.88	253.47	0.00	0.00	3g	2	65.48	1,396.97		2,971	4,621
02310		**Grading**																		
02315		**Excavation & Fill**			13															
SC	2	Drilling & Blasting (Seepage Canal	1,215,573	CY	6.02	B6	0.080	97,246	4,051.9	17.11	1,664,200.93	1.89	2,297,433.60	6h	4	34.56	3,361,252.76		0	7,322,887
SC	2	Excavating Cap Rock (Seepage C	1,215,573	CY	1.75	B2	0.015	17,975	1,123.5	16.37	294,255.51	0.00	0.00	1b	1	102.03	1,834,074.88		0	2,128,330
SC	2	Dump Truck (Canal/Stock Pile Are	1,519,467	CY	1.33	C1	0.013	19,584	2,448.0	11.88	232,660.74	0.00	0.00	3e1	1	91.31	1,788,312.15		0	2,020,973
SC	2	Hoe Ram (Stock Pile Area)	159,544	CY	1.74	A10	0.077	12,285	1,228.5	17.89	219,776.65	0.00	0.00	6i	1	5.91	58,102.61		0	277,879
SC	2	Dozer Angle Blade (Stock Pile Are	1,519,467	CY	1.20	B2	0.016	23,932	1,495.7	16.37	391,760.29	0.00	0.00	6c	1	120.02	1,436,178.29		0	1,827,939
					13.32															
SC	3	Excavated - Silty, Sand, Shells (Se	958,372	CY	0.80	B3	0.005	4,792	199.7	16.75	80,263.65	0.00	0.00	1n	1	144.10	690,522.06		0	770,786
SC	3	Cut Through Limestone	113,846	CY	4.02	B3	0.025	2,846	118.6	16.75	47,673.03	0.00	0.00	1n	1	144.10	410,139.31		0	457,812
SC	3	Haul to Dewater & Work Stock Pile	1,179,440	CY	1.33	C1	0.013	15,202	1,900.2	11.88	180,596.82	0.00	0.00	3e1	1	91.31	1,388,122.91		0	1,568,719
SC	3	Dozer Angle Blade - Work Stock P	1,179,440	CY	0.69	B2	0.009	10,615	663.4	16.37	173,766.86	0.00	0.00	6c	1	120.02	637,022.70		0	810,790
EM	4	Drilling & Blasting (Prod. Blast Are	2,912,398	CY	6.02	B6	0.080	232,992	9,708.0	17.11	3,987,267.07	1.89	5,504,432.28	6h	4	34.56	8,053,241.75		0	17,544,941
EM	4	Excavating Cap Rock (Prod. Blast	2,912,398	CY	1.75	B2	0.015	43,067	2,691.7	16.37	705,008.20	0.00	0.00	1b	1	102.03	4,394,268.88		0	5,099,277
EM	4	Dump Truck (Prod. Blast Stock Pile	3,640,498	CY	1.71	C1	0.017	60,328	7,541.0	11.88	716,699.55	0.00	0.00	3e1	1	91.31	5,508,804.50		0	6,225,504
EM	4	Hoe Ram (Stock Pile Area)	382,252	CY	1.74	A10	0.077	29,433	2,943.3	17.89	526,563.93	0.00	0.00	6i	1	5.91	139,208.32		0	665,772
EM	4	Dozer Angle Blade (Prod. Blast Ar	3,640,498	CY	1.20	B2	0.016	57,338	3,583.6	16.37	938,620.38	0.00	0.00	6c	1	120.02	3,440,946.52		0	4,379,567
EM	4	Excavated - Silty, Sand, Shells (Pr	4,732,360	CY	0.80	B3	0.005	23,662	985.9	16.75	396,335.17	0.00	0.00	1n	1	144.10	3,409,739.96		0	3,806,075
EM	4	Haul to Dewater & Work Stock Pile	5,205,596	CY	1.71	C1	0.017	86,264	10,783.0	11.88	1,024,818.29	0.00	0.00	3e1	1	91.31	7,877,113.36		0	8,901,932
EM	4	Dozer Angle Blade (Work Stock Pil	5,205,596	CY	0.69	B2	0.009	46,850	2,928.1	16.37	766,940.49	0.00	0.00	6c	1	120.02	2,811,574.59		0	3,578,515
EM	4 W/O	Drilling & Blasting (Prod. Blast Are	94,664	CY	6.02	B6	0.080	7,573	315.5	17.11	129,601.45	1.89	178,915.13	6h	4	34.56	261,761.20		0	570,278
EM	4 W/O	Excavating Cap Rock (Prod. Blast	94,664	CY	1.75	B2	0.015	1,400	87.5	16.37	22,915.47	0.00	0.00	1b	1	102.03	142,830.57		0	165,746

						Labor						Material		Equipment						
CSI Div. / Sect.	DESCRIPTION		Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost
EM	4 WIO	Dozer Angle Blade (Prod. Blast Ar	113,597	CY	1.20	B2	0.016	1,789	111.8	16.37	29,288.41	0.00	0.00	6c	1	120.02	107,370.18		0	136,659
EM	4 WIO	Excavated - Silty, Sand, Shells (Pr	268,215	CY	0.80	B3	0.005	1,341	55.9	16.75	22,463.00	0.00	0.00	1n	1	144.10	193,253.06		0	215,716
EM	4 WIO	Haul to Dewater & Work Stock Pile	295,036	CY	1.71	C1	0.017	4,889	611.1	11.88	58,083.39	0.00	0.00	3e1	1	91.31	446,449.38		0	504,533
EM	4 WIO	Dozer Angle Blade (Work Stock Pi	295,036	CY	0.69	B2	0.009	2,655	166.0	16.37	43,467.71	0.00	0.00	6c	1	120.02	159,350.98		0	202,819
EM	5	Crusher (Level Coarse Borrow Are	126,034	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,287,510	2,287,510
EM	5	Dozer Loader (Level Coarse Borro	126,034	CY	0.82	B5	0.013	1,607	133.9	17.22	27,676.66	0.00	0.00	6j	1	47.46	76,259.36		0	103,936
EM	5	Dump Truck (Level Coarse Borrow	176,447	CY	1.71	C1	0.017	2,924	365.5	11.88	34,736.88	0.00	0.00	3e1	1	91.31	266,999.84		0	301,737
EM	5	Dozer Angle Blade (New Embankm	176,447	CY	1.03	B2	0.014	2,382	148.9	16.37	38,993.91	0.00	0.00	6c	1	120.02	142,950.20		0	181,944
EM	5	Cruncher (Transition Borrow Area)	210,487.200	CY	18.15	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		3,820,343	3,820,343
EM	5	Dozer Loader (Transition Borrow A	210,487.200	CY	0.82	B5	0.013	2,684	223.6	17.22	46,222.46	0.00	0.00	6j	1	47.46	127,359.83		0	173,582
EM	5	Dump Truck (Transition Borrow Are	294,682.080	CY	1.71	C1	0.017	4,883	610.4	11.88	58,013.64	0.00	0.00	3e1	1	91.31	445,913.22		0	503,927
EM	5	Backhoe (Vertical New Embankm	294,682.080	CY	3.28	B4a	0.153	45,086	939.3	17.08	770,150.14	0.00	0.00	2	1	4.33	195,186.36		0	965,337
EM	5	Crusher (Granular Toe Borrow Are	114,188	CY	18.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		2,112,477	2,112,477
EM	5	Dozer Loader (Granular Toe Borro	114,188	CY	0.82	B5	0.013	1,456	121.3	17.22	25,075.38	0.00	0.00	6j	1	47.46	69,091.87		0	94,167
EM	5	Dump Truck (Granular Toe Borrow	159,863	CY	1.71	C1	0.017	2,649	331.1	11.88	31,472.02	0.00	0.00	3e1	1	91.31	241,904.98		0	273,377
EM	5	Backhoe Loader (New Embankme	159,863	CY	3.84	B4-	0.160	25,578	1,065.8	16.65	425,960.53	0.00	0.00	2a	1	7.33	187,524.25		0	613,485
EM	5	Compacting (New Embankment Ar	159,863	CY	1.79	A1	0.089	14,228	1,778.5	17.51	249,129.04	0.00	0.00	8b2	1	2.64	37,595.51		0	286,725
EM	6	Clean Cap Rock Surface (Embank	163,419	SY	0.87	C1a	0.030	4,903	204.3	15.63	76,643.34		0.00	3j4	5	13.25	64,953.03		0	141,596
EM	6	Cement Grout Cap Rock (Embank	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0
EM	6 WIO	Clean Cap Rock Surface (Embank	267,960	SY	0.87	C1a	0.030	8,039	335.0	15.63	125,673.24		0.00	3j4	5	13.25	106,504.46		0	232,178
EM	6 WIO	Cement Grout Cap Rock (Embank	0	CY	0.00	B4	1.500	0	0.0	16.87	0.00	45.00	0.00	3i	2	5.78	0.00		0	0
EM	6 w/o	Lean Concrete Fill In Cap Rock Cu	91,168	CY	67.54	D8	0.137	12,490	260.2	17.69	220,885.93	65.00	5,925,920.00	8b1	2	0.83	10,376.69		0	6,157,183
EM	6 w/o	Concrete Batch Plant & Delivery	93,903	CY	26.15	C6a	0.480	45,073	704.3	14.44	650,635.38	0.00	0.00	8h1	6	40.04	1,804,643.90		0	2,455,279
EM	6 w/o	Cut Off Wall (Embankment Area)	2,051,280	SF	2.50	B2	0.000	0	0.0	16.37	0.00	0.00	0.00				0.00	5,128,200	0	5,128,200
SC	6 w/o	Cut Off Wall (Through Limestone)	15,043	CY	45.00	B2	0.000	0	0.0	16.37	0.00		0.00				0.00	676,922	0	676,922
EM	7	Dozer Loader (Sand Horiz. Filter)	198,290	CY	0.46	B2	0.006	1,190	74.4	16.37	19,476.08	0.00	0.00	6c	1	120.02	71,398.58		0	90,875
EM	7	Dump Truck (Sand Horiz. Filter)	198,290	CY	0.93	C1	0.009	1,785	223.1	11.88	21,201.21	0.00	0.00	3e1	1	91.31	162,959.94		0	184,161
EM	7	Wash & Screen (Sand Horiz. Filter	198,290	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	779,281	0	779,281
EM	7	Dozer Angle Blade (Sand Horiz. Fil	198,290	CY	0.69	B2	0.009	1,785	111.5	16.37	29,214.12	0.00	0.00	6c	1	120.02	107,097.87		0	136,312
EM	7	Dozer & Dump Trucks (Sand Horiz	198,290	CY	1.75	C6	0.024	4,660	166.4	14.17	66,029.71	0.00	0.00	3f1	2	60.38	281,370.11		0	347,400
EM	7	Dozer Angle Blade (Sand Horiz. Fil	198,290	CY	0.69	B2	0.009	1,785	111.5	16.37	29,214.12	0.00	0.00	6c	1	120.02	107,097.87		0	136,312
EM	7	Compact (Sand Horiz. Filter)	198,290	CY	0.79	B5	0.009	1,785	148.7	17.22	30,736.99	0.00	0.00	6g1	1	70.54	125,889.60		0	156,627
EM	7	Dozer Loader (Sand Vert. Filter)	136,752	CY	0.46	B2	0.006	821	51.3	16.37	13,431.78	0.00	0.00	6c	1	120.02	49,240.40		0	62,672
EM	7	Dump Truck (Sand Vert. Filter)	136,752	CY	0.93	C1	0.009	1,231	153.8	11.88	14,621.52	0.00	0.00	3e1	1	91.31	112,386.17		0	127,008
EM	7	Wash & Screen (Sand Vert. Filter)	136,752	CY	3.93		0.000	0	0.0	11.88	0.00	0.00	0.00				0.00	537,435	0	537,435
EM	7	Dozer Angle Blade (Sand Vert. Filt	136,752	CY	0.69	B2	0.009	1,231	76.9	16.37	20,147.67	0.00	0.00	6c	1	120.02	73,860.60		0	94,008
EM	7	Dozer & Dump Trucks (Sand Vert.	136,752	CY	1.75	C6	0.024	3,214	114.8	14.17	45,537.73	0.00	0.00	3f1	2	60.38	194,048.35		0	239,586
EM	7	Backhoe (Sand Vert. Filter)	136,752	CY	3.28	B4a	0.153	20,923	435.9	17.08	357,400.67	0.00	0.00	2	1	4.33	90,579.40		0	447,980
EM	8	Dozer & Dump Trucks (Max. 6" Ra	1,490,597	CY	1.75	C6	0.024	35,029	1,251.0	14.17	496,361.28	0.00	0.00	3f1	2	60.38	2,115,127.03		0	2,611,488
EM	8	Dozer With Ripper Attach. (Max. 6"	1,490,597	CY	2.04	B5	0.020	29,812	2,484.3	17.22	513,460.91	0.00	0.00	6g	2	84.81	2,528,406.98		0	3,041,868
EM	8	Dozer Angle Blade (Max. 6" Rando	1,490,597	CY	0.69	B2	0.009	13,415	838.5	16.37	219,609.63	0.00	0.00	6c	1	120.02	805,080.52		0	1,024,690
EM	8	Compact (Max. 6" Random Fill)	1,490,597	CY	0.79	B5	0.009	13,415	1,117.9	17.22	231,057.41	0.00	0.00	6g1	1	70.54	946,342.51		0	1,177,400
EM	8 WIO	Dozer & Dump Trucks (Max. 6" Ra	1,496,880	CY	1.75	C6	0.024	35,177	1,256.3	14.17	498,453.56	0.00	0.00	3f1	2	60.38	2,124,042.76		0	2,622,496
EM	8 WIO	Dozer With Ripper Attach. (Max. 6"	1,496,880	CY	2.04	B5	0.020	29,938	2,494.8	17.22	515,625.26	0.00	0.00	6g	2	84.81	2,539,064.79		0	3,054,690
EM	8 WIO	Dozer Angle Blade (Max. 6" Rando	1,496,880	CY	0.69	B2	0.009	13,472	842.0	16.37	220,535.33	0.00	0.00	6c	1	120.02	808,474.11		0	1,029,009
EM	8 WIO	Compact (Max. 6" Random Fill)	1,496,880	CY	0.79	B5	0.009	13,472	1,122.7	17.22	232,031.37	0.00	0.00	6g1	1	70.54	950,331.56		0	1,182,363
EM	8	Dozer & Dump Trucks (Mass Rand	2,062,904	CY	1.75	C6	0.024	48,478	1,731.4	14.17	686,936.69	0.00	0.00	3f1	2	60.38	2,927,219.38		0	3,614,156
EM	8	Dozer Angle Blade (Mass Random	2,062,904	CY	0.69	B2	0.009	18,566	1,160.4	16.37	303,927.63	0.00	0.00	6c	1	120.02	1,114,187.12		0	1,418,115
EM	8	Compact (Mass Random Fill)	2,062,904	CY	0.79	B5	0.009	18,566	1,547.2	17.22	319,770.74	0.00	0.00	6g1	1	70.54	1,309,685.95		0	1,629,457
EM	8 WIO	Dozer & Dump Trucks (Mass Rand	1,071,840	CY	1.75	C6	0.024	25,188	899.6	14.17	356,917.36	0.00	0.00	3f1	2	60.38	1,520,919.51		0	1,877,837
EM	8 WIO	Dozer Angle Blade (Mass Random	1,071,840	CY	0.69	B2	0.009	9,647	602.9	16.37	157,914.19	0.00	0.00	6c	1	120.02	578,907.39		0	736,822
EM	8 WIO	Compact (Mass Random Fill)	1,071,840	CY	0.79	B5	0.009	9,647	803.9	17.22	166,145.92	0.00	0.00	6g1	1	70.54	680,484.33		0	846,630
EM	8	Dozer Loader (Rock Fill Borrow Ar	2,602,908	CY	1.15	B2	0.015	39,044	2,440.2	16.37	639,144.06	0.00	0.00	6c	1	120.02	2,343,077.75		0	2,982,222
EM	8	Dump Truck (Rock Fill Borrow Are	3,644,071	CY	0.93	C1	0.009	32,797	4,099.6	11.88	389,624.09	0.00	0.00	3e1	1	91.31	2,994,787.64		0	3,384,412
EM	8	Dozer Angle Blade (New Embankm	3,644,071	CY	1.03	B2	0.014	49,195	3,074.7	16.37	805,321.51	0.00	0.00	6c	1	120.02	2,952,277.97		0	3,757,599
EM	10	Common Borrow (Top Soil - Peat)	225,641	CY	3.65	C6	0.047	10,605	378.8	14.17	150,274.52	0.00	0.00	3f	3	63.57	674,182.48		0	824,457
EM	10 WIO	Common Borrow (Top Soil - Peat)	97,020	CY	3.65	C6	0.047	4,560	162.9	14.17	64,614.35	0.00	0.00	3f	3	63.57	289,881.90		0	354,496
02370	**Erosion & Sedimentation Control**																			
02600	**Drainage & Containment**																			
02620	**Subdrainage System (Seepage Water)**																			
02700	**Bases, Ballasts, Pavements & Appurtenances**																			
EM	8 w/o	Crusher (Transition Borrow Area)	74177	CY	12.50	B2		0	0.0	16.37	0.00	0.00	0.00				0.00		927,217	927,217
EM	8 w/o	Aggregate Base 1-1/2" Stone, 12"	222532	SY	1.13	B7a	0.019	4,256	59.1	16.28	69,281.74	0.00	0.00	7a+	7	42.93	182,704.54		0	251,986
02910	**Plant Preparation**																			

					Labor						Material		Equipment						
CSI Div. / Sect.	DESCRIPTION	Quantity	Unit	Unit Cost	Crew Code	M-H per Unit	Man Hours	Duration Days	Average Wage Rate	Labor Cost	Unit Cost	Material Cost	Code	No.	Avg. Cost ($/hr)	Equipment Cost	Sub-contract	Other	Total Cost
EM	10 Fine Grading	670,085	SY	0.32	B6a	0.008	5,361	335.0	17.30	92,712.93	0.00	0.00	7d	1	22.97	123,136.33		0	215,849
EM	10 W/O Fine Grading	291,060	SY	0.32	B6a	0.008	2,328	145.5	17.30	40,271.06	0.00	0.00	7d	1	22.97	53,485.86		0	93,757
02920	**Lawns & Grasses**																		
EM	10 Hydro or Air Seeding w/ Mulch & F	670,085	SY	0.24	C4a	0.003	2,010	3.0	15.49	31,138.84	0.15	100,512.72	3j5	2	14.11	28,360.99		0	160,013
EM	10 W/O Hydro or Air Seeding w/ Mulch & F	291,060	SY	0.24	C4a	0.003	873	3.0	15.49	13,525.56	0.15	43,659.00	3j5	2	14.11	12,318.96		0	69,504
	Subtotal Site Construction						1,562,823			$24,644,842		$14,050,873				$96,132,282	$7,121,839	$9,250,795	$151,200,631
3	**Concrete**																		
03050	**Basic Concrete Materials & Methods**																		
03100	**Concrete & Forms & Accessories**																		
03200	**Concrete Reinforcement**																		
03300	**Cast-in-Place Concrete**																		
03310	**Structural concrete**																		
	Roller Compacted Concrete																		
EM	11 Mass Placement, 1' Lift, 12" Layer	205,128	CY	1.07	B5	0.009	1,846	76.9	16.91	31,212.28		0.00	6f1	2	102.16	188,599.39		0	219,812
EM	11 Sloped Face, Nonformed, 1' Lift	218,803	CY	5.00	B5	0.042	9,190	382.9	16.91	155,367.78		0.00	6f1	2	102.16	938,805.85		0	1,094,174
EM	11 Roller Compacted Concrete, 1.5"-2	423,931	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	19,076,904.00				0.00		0	19,076,904
EM	11 Dump Truck (18 CY) Conveying M	423,931	CY	1.24	C1	0.012	5,087	635.9	11.88	60,435.63		0.00	3e1	1	91.31	464,529.50		0	524,965
EM	11 Truck Mtd. Hydraulic Crane 100 Tc	205,128	CY	0.59	D18a	0.025	5,128	128.2	17.43	89,405.04		0.00	8g	4	6.32	32,389.38		0	121,794
EM	11 Surface Prep. Vacuum Truck	307,692	SY	0.18	C6a-	0.006	1,846	92.3	17.72	32,710.12		0.00	3j2	1	12.86	23,734.74		0	56,445
EM	11 Surface Prep. Water Clean	307,692	SY	0.22	C1a	0.008	2,462	102.6	15.63	38,482.01		0.00	3j3	4	11.43	28,138.83		0	66,621
EM	11 Surface Prep. Water Blast	307,692	SY	0.87	C1a	0.030	9,231	384.6	15.63	144,307.55		0.00	3j4	5	13.25	122,296.50		0	266,604
EM	11 Concrete Batch Plant & Delivery	436,649	CY	26.15	C6a	0.480	209,592	3,274.9	14.44	3,025,454.53	0.00	0.00	6h1	6	40.04	8,391,594.14		0	11,417,049
EM	11 W/O Mass Placement, 1' Lift, 12" Layer	138,600	CY	1.07	B5	0.009	1,247	52.0	16.91	21,089.36		0.00	6f1	2	102.16	127,432.02		0	148,521
EM	11 W/O Vertical Face, Formed, 1' Lift	147,840	CY	7.14	B5	0.060	8,870	369.6	16.91	149,968.90		0.00	6f1	2	102.16	906,183.25		0	1,056,152
EM	11 W/O Roller Compacted Concrete, 1.5"-2	286,440	CY	45.00	B5	0.000	0	0.0	16.91	0.00	45.00	12,889,800.00				0.00		0	12,889,800
EM	11 W/O Dump Truck (18 CY) Conveying M	286,440	CY	1.24	C1	0.012	3,437	429.7	11.88	40,834.89		0.00	3e1	1	91.31	313,871.28		0	354,706
EM	11 W/O Truck Mtd. Hydraulic Crane 100 Tc	286,440	CY	0.44	D18a	0.025	7,161	179.0	17.43	124,844.87		0.00	8g	0	0.00	0.00		0	124,845
EM	11 W/O Surface Prep. Vacuum Truck	207,900	SY	0.11	C6a-	0.006	1,247	62.4	17.72	22,101.43		0.00	3j2	0	0.00	0.00		0	22,101
EM	11 W/O Surface Prep. Water Clean	207,900	SY	0.22	C1a	0.008	1,663	69.3	15.63	26,001.36		0.00	3j3	4	11.43	19,012.72		0	45,014
EM	11 W/O Surface Prep. Water Blast	207,900	SY	0.87	C1a	0.030	6,237	259.9	15.63	97,505.10		0.00	3j4	5	13.25	82,632.77		0	180,138
EM	11 W/O Concrete Batch Plant & Delivery	295,033	CY	26.15	C6a	0.480	141,616	2,212.7	14.44	2,044,226.04	0.00	0.00	6h1	6	40.04	5,669,996.04		0	7,714,222
03400	**Precast Concrete**																		
03500	**Cementitious Decks & Underlay**																		
03600	**Grouts**																		
03900	**Concrete Restorations & Cleaning**																		
		0	LS	0	A1	0.000	0	0.0	17.51	0.00	0.00	0.00	1	1	65.45	0.00		0	0
	Subtotal Concrete						415,861			$6,103,947		$31,966,704				$17,309,216	$0	$0	$55,379,867
	Construction Subtotal (Direct Costs)						1,978,684			$30,748,789		$46,017,577				$113,441,498	$7,121,839	$39,204,968	$236,534,671

Indirect Costs			
Sales Tax	6%	of purchased materials + Rental Equipment	9,567,544
Overhead and Profit	17%	of construction cost + general requirements	39,534,552
Bonds and Insurance	3.5%	of construction cost + general requirements + sales tax + overhead and profit	9,997,287
Project Reserve	5%	of construction cost	14,781,703
Contingency	20%	of construction cost + general conditions + sales tax + overhead and profit + bonds and insurance + escalation	62,083,151
Construction Subtotal Indirects			$135,964,237
Total Construction (directs and Indirects)			$372,498,908

Table 23.5-2 Opinion of Probable Construction Cost - 3,600 CFS Northeast Pump Station

BODR SUBMITTAL

EAA Reservoir A-1
3,600 CFS Northeast Pump Station

OPINION OF
PROBABLE CONSTRUCTION COST
October 6, 2005

<u>SUMMARY</u>

General Requirements		$5,500,000
Sitework		2,500,000
Pump Station		59,100,000
Project Reserve	5%	3,300,000
Contingencies	20%	14,100,000
Mid-Point of Construction		0
Rate = % 4%		
Time = Years 0		
TOTAL PROBABLE CONSTRUCTION COST		**$84,500,000**

BLACK and VEATCH

3,600 CFS Northeast Pump Station
Probable Construction Cost
October 6, 2005

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
GENERAL REQUIREMENTS				
Mobilization		Lump Sum		986,400
Supervision		Lump Sum		3,082,400
Temporary facilities		Lump Sum		739,800
Temporary utilities		Lump Sum		493,200
Equipment rental and misc.		Lump Sum		246,600
Total - General Requirements				$5,548,000
SITEWORK				
Earthwork				
Site excavation				
Peat	9,560	cu yd	5.15	49,200
Caprock	28,420	cu yd	28.67	814,800
Silty Sandy Shells	65,210	cu yd	4.29	279,600
Embankment	14,700	cu yd	10.74	157,900
Wasting (off site)	117,890	cu yd	2.14	252,000
Seepage Culvert	610	lin ft	680.00	414,800
Finish grading	14,210	sq yd	1.10	15,600
Surfacings				
Concrete pavement	0	sq yd	60.00	0
Helicopter pad	225	cu yd	350.00	78,800
Asphalt pavement	9,889	sq yd	35.00	346,100
Concrete curb and gutter	2,000	lin ft	25.00	50,000
Marking and signage		Lump Sum		4,000
Fencing				
Chain link	1,550	lin ft	25.00	38,800
Gates	4	each	2,500.00	10,000
Storm drainage				
Catch basins	2	each	1,200.00	2,400
Yard structures				
Entrance gate		Lump Sum		3,000
Total - Sitework				$2,517,000

BLACK and VEATCH

3,600 CFS Northeast Pump Station
Probable Construction Cost
October 6, 2005

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
PUMPING STATION TYPE STRUCTURE				
Earthwork				
Structural excavation				
Peat	6,080	cu yd	5.15	31,300
Caprock	18,240	cu yd	28.67	523,000
Silty Sandy Shells	50,240	cu yd	4.29	215,400
Embankment	14,700	cu yd	10.74	157,800
Compacted fill	1,520	cu yd	20.00	30,400
Granular fill	1,035	cu yd	45.00	46,600
Sheeting, shoring and dewatering				
Sheet Pile	27,000	sq ft	25.00	675,000
Sheet Pile (Rental)	83,250	sq ft	15.00	1,248,800
Dewatering		Lump Sum		754,000
Concrete, cast in place				
Slab on grade/footings	10,831	cu yd	450.00	4,874,000
Walls	11,428	cu yd	720.00	8,228,300
Suspended	3,800	cu yd	820.00	3,116,000
Equipment Bases	60	cu yd	350.00	21,000
Embedded accessories		Lump Sum		812,000
Concrete, precast				
Tilt up panels	47,328	sq ft	8.97	424,700
Double tee roof	34,560	sq ft	7.54	260,500
Masonry				
Concrete block				
8" Wall	8,670	sq ft	18.00	156,100
Bond beam	564	lin ft	22.00	12,400
Metal				
Structural steel	600	ton	4,000.00	2,400,000
Stairway		Lump Sum		15,000
Handrail	600	lin ft	45.00	27,000
Trash Rack	8,832	sq ft	41.00	362,100
Access hatch	2	each	2,400.00	4,800
Ladder		Lump Sum		6,300
Guard post	6	each	450.00	2,700
Miscellaneous		Lump Sum		86,200
Thermal and moisture protection				
Built-up roofing	34,560	sq ft	4.00	138,200
Cant and counterflashing	816	lin ft	25.00	20,400
Concrete dampproofing	56,700	sq ft	1.00	56,700
Caulking, flashing, etc.		Lump Sum		4,100
Doors				
Hollow metal	144	sq ft	55.00	7,900
Rolling steel	2	each	7,000.00	14,000
Windows	300	sq ft	45.00	13,500
Painting		Lump Sum		16,000
Miscellaneous specialties		Lump Sum		1,800

BLACK and VEATCH

3,600 CFS Northeast Pump Station
Probable Construction Cost
October 6, 2005

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
Equipment				
Pumps (600 cfs)	6	each	3,530,000.00	21,180,000
Seepage Pumps (150 cfs)	3	each	490,000.00	1,470,000
Exhaust system	6	each	44,000.00	264,000
Cooling water system	6	each	32,000.00	192,000
Service water system	1	each	45,000.00	45,000
Pump lube water system	1	each	50,000.00	50,000
Compressed air system	1	each	80,000.00	80,000
Vacuum system	1	each	170,000.00	170,000
Fuel oil system	6	each	104,000.00	624,000
Lube oil system	1	each	60,000.00	60,000
Waste lube oil system	1	each	30,000.00	30,000
Sanitary waste system		Lump Sum		20,000
Automatic Trash rake		Lump Sum		475,000
Ventilation fans	8	each	18,125.00	145,000
Sump pumps, duplex				
50 gpm @ 25' RTH		Lump Sum		16,000
Instrumentation		Lump Sum		1,241,000
Conveying system				
Bridge Crane		Lump Sum		165,000
Mechanical				
Heating, ventilating and AC			(Listed separately)	
Plumbing		Lump Sum		0
Potable water system		Lump Sum		90,000
Process piping				
Steel pipe	40,000	pound	3.77	150,900
Electric actuator				
15' X 21'	6	each	220,000.00	1,320,000
16' X 21'	6	each	220,000.00	1,320,000
8' X 8'	6	each	50,000.00	300,000
Electrical				
Generator		Lump Sum		1,240,000
Electrical Labor and Materials		Lump Sum		3,720,000
Total -				$59,131,000

Table 23.5-3 Opinion of Probable Construction Cost Southeast Gate Structure

BODR SUBMITTAL

EAA Reservoir A-1

Southeast Gate Structure
OPINION OF
PROBABLE CONSTRUCTION COST
October 6, 2005

SUMMARY

General Requirements			$338,000
Sitework			440,000
Gate Structure			3,312,000
Project Reserve		5%	205,000
Contingencies		30%	1,289,000
Mid-Point of Construction			0
Rate = %	4%		
Time = Years	0		
TOTAL PROBABLE CONSTRUCTION COST			**$5,580,000**
ENGINEERING (Final Design and CPS)		0%	0
TOTAL PROBABLE PROJECT COST			**$5,580,000**

BLACK & VEATCH

Probable Construction Cost
October 6, 2005

Southeast Gate Structure

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
GENERAL REQUIREMENTS				
Mobilization		Lump Sum		60,000
Supervision		Lump Sum		187,600
Temporary facilities		Lump Sum		45,000
Temporary utilities		Lump Sum		30,000
Equipment rental & misc.		Lump Sum		15,000
Total - General Requirements				$338,000
SITEWORK				
Demolition & removal		Lump Sum		0
Clear and grub	1	acre	4,187.00	4,200
Earthwork				
Site excavation				
Peat	4,450	cu yd	5.15	22,900
Caprock	11,290	cu yd	28.67	323,700
Silty Sandy Shells	8,210	cu yd	4.29	35,200
Wasting (off site)	23,950	cu yd	2.14	51,200
Finish grading	2,225	sq yd	1.10	2,400
Total - Sitework				$440,000
GATE STRUCTURE				
Earthwork				
Structural excavation				
Peat	1,200	cu yd	5.15	6,200
Caprock	3,610	cu yd	28.67	103,500
Silty Sandy Shells	3,610	cu yd	4.29	15,500
Compacted fill	1,100	cu yd	20.00	22,000
Granular fill	300	cu yd	45.00	13,500
Sheeting, shoring & dewatering				
Sheet Pile	1,560	sq ft	25.00	39,000
Sheet Pile (Rental)	25,730	sq ft	15.00	386,000
Dewatering		Lump Sum		125,000
Concrete, cast in place				
Slab on grade/footings	760	cu yd	450.00	342,000
Slab on grade/apron	440	cu yd	450.00	198,000
Walls	590	cu yd	720.00	424,800
Wing walls	360	cu yd	720.00	259,200
Suspended	570	cu yd	820.00	467,400
Actuator supports	50	cu yd	820.00	41,000
Concrete lining of canal outlet	250	cu yd	450.00	112,500
Embedded accessories		Lump Sum		60,600
Metal				

BLACK & VEATCH

Probable Construction Cost
October 6, 2005

Southeast Gate Structure

Item Description	Quantity	Unit	Unit Cost	Total Cost
			$	$
Handrail	310	lin ft	45.00	14,000
Trash Rack	1,100	sq ft	41.00	45,100
Miscellaneous		Lump Sum		11,800
Equipment				
Mechanical				
Electric actuated gates				
10' X 10'	5	each	75,000.00	375,000
6' X 6'	0	each	50,000.00	0
Dewatering bulkheads	5	each	35,000.00	175,000
Electrical				
Electrical Labor & Materials		Lump Sum		75,000
Total - Gate Structure				$3,312,000

Table 23.5-4 **Opinion of Probable Construction Cost Southwest Gate Structure**

BODR SUBMITTAL

EAA Reservoir A-1

Southwest Gate Structure
OPINION OF
PROBABLE CONSTRUCTION COST
October 6, 2005

SUMMARY

General Requirements			$431,000
Sitework			529,000
Gate Structure			4,262,000
Project Reserve		5%	261,000
Contingencies		30%	1,645,000
Mid-Point of Construction			0
Rate = %	4%		
Time = Years	0		
TOTAL PROBABLE CONSTRUCTION COST			**$7,130,000**
ENGINEERING (Final Design and CPS)		0%	0
TOTAL PROBABLE PROJECT COST			**$7,130,000**

BLACK & VEATCH

Probable Construction Cost
October 6, 2005

Southwest Gate Structure

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
GENERAL REQUIREMENTS				
Mobilization		Lump Sum		76,700
Supervision		Lump Sum		239,600
Temporary facilities		Lump Sum		57,500
Temporary utilities		Lump Sum		38,300
Equipment rental & misc.		Lump Sum		19,200
Total - General Requirements				$431,000
SITEWORK				
Demolition & removal		Lump Sum		0
Clear and grub	1	acre	4,187.00	4,200
Earthwork				
Site excavation				
Peat	5,190	cu yd	5.15	26,700
Caprock	13,520	cu yd	28.67	387,600
Silty Sandy Shells	10,440	cu yd	4.29	44,800
Wasting (off site)	29,160	cu yd	2.14	62,400
Finish grading	2,595	sq yd	1.10	2,900
Total - Sitework				$529,000
GATE STRUCTURE				
Earthwork				
Structural excavation				
Peat	1,600	cu yd	5.15	8,200
Caprock	4,810	cu yd	28.67	137,900
Silty Sandy Shells	4,810	cu yd	4.29	20,600
Compacted fill	1,100	cu yd	20.00	22,000
Granular fill	400	cu yd	45.00	18,000
Sheeting, shoring & dewatering				
Sheet Pile	2,080	sq ft	25.00	52,000
Sheet Pile (Rental)	27,500	sq ft	15.00	412,500
Dewatering		Lump Sum		170,000
Concrete, cast in place				
Slab on grade/footings	1,020	cu yd	450.00	459,000
Slab on grade/apron	590	cu yd	450.00	265,500
Walls	780	cu yd	720.00	561,600

Wing walls	360	cu yd	720.00	259,200
Suspended	760	cu yd	820.00	623,200
Actuator supports	70	cu yd	820.00	57,400
Concrete lining of canal outlet	330	cu yd	450.00	148,500
Embedded accessories		Lump Sum		77,900
Metal				
Handrail	360	lin ft	45.00	16,200
Trash Rack	1,500	sq ft	41.00	61,500
Miscellaneous		Lump Sum		15,500
Equipment				
Mechanical				
Electric actuated gates				
10' X 10'	7	each	75,000.00	525,000
6' X 6'	0	each	50,000.00	0
Dewatering bulkheads	7	each	35,000.00	245,000
Electrical				
Electrical Labor & Materials		Lump Sum		105,000
Total - Gate Structure				$4,262,000

Table 23.5-5 Opinion of Probable Construction Cost Northeast Gate Structure

BODR SUBMITTAL

EAA Reservoir A-1

Northeast Gate Structure
OPINION OF
PROBABLE CONSTRUCTION COST
October 6, 2005

SUMMARY

General Requirements		$378,000
Sitework		470,000
Gate and Spillway Structure		3,727,000
Project Reserve	5%	229,000
Contingencies	30%	1,441,000
Mid-Point of Construction		0
Rate = % 4%		
Time = Years 0		
TOTAL PROBABLE CONSTRUCTION COST		**$6,250,000**
ENGINEERING (Final Design and CPS)	0%	0
TOTAL PROBABLE PROJECT COST		**$6,250,000**

BLACK & VEATCH

Probable Construction Cost
October 6, 2005

Northeast Gate Structure

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
GENERAL REQUIREMENTS				
Mobilization		Lump Sum		67,200
Supervision		Lump Sum		209,900
Temporary facilities		Lump Sum		50,400
Temporary utilities		Lump Sum		33,600
Equipment rental & misc.		Lump Sum		16,800
Total - General Requirements				$378,000
SITEWORK				
Demolition & removal		Lump Sum		0
Clear and grub	1	acre	4,187.00	4,200
Earthwork				
Site excavation				
Peat	4,710	cu yd	5.15	24,300
Caprock	12,060	cu yd	28.67	345,800
Silty Sandy Shells	8,980	cu yd	4.29	38,500
Wasting (off site)	25,750	cu yd	2.14	55,000
Finish grading	2,355	sq yd	1.10	2,600
Total - Sitework				$470,000
GATE STRUCTURE				
Earthwork				
Structural excavation				
Peat	1,313	cu yd	5.15	6,800
Caprock	3,743	cu yd	28.67	107,300
Silty Sandy Shells	3,743	cu yd	4.29	16,000
Compacted fill	1,100	cu yd	20.00	22,000
Granular fill	275	cu yd	45.00	12,400
Sheeting, shoring & dewatering				
Sheet Pile	1,740	sq ft	25.00	43,500
Sheet Pile (Rental)	26,140	sq ft	15.00	392,100
Dewatering		Lump Sum		145,000
Concrete, cast in place				
Slab on grade/footings	785	cu yd	450.00	353,300
Slab on grade/apron	455	cu yd	450.00	204,800
Walls	615	cu yd	720.00	442,800

Wing Walls	360	cu yd	720.00	259,200
Suspended	590	cu yd	820.00	483,800
Actuator Supports	50	cu yd	820.00	41,000
Concrete lining of canal outlet	270	cu yd	450.00	121,500
Embedded accessories		Lump Sum		62,500
Metal				
Handrail	330	lin ft	45.00	14,900
Trash Rack	1,200	sq ft	41.00	49,200
Miscellaneous	1	Lump Sum		12,800
Equipment				
Mechanical				
Electric actuated gates				
10' X 10'	5	each	75,000.00	375,000
Dewatering bulkheads	6	each	35,000.00	210,000
Electrical				
Electrical Labor & Materials		Lump Sum		75,000
SPILLWAY STRUCTURE				
Earthwork				
Structural excavation				
Peat	27	cu yd	5.15	100
Caprock	277	cu yd	28.67	7,900
Silty Sandy Shells	277	cu yd	4.29	1,200
Granular fill	25	cu yd	45.00	1,100
Concrete, cast in place				
Slab on grade/footings	65	cu yd	450.00	29,300
Slab on grade/apron	35	cu yd	450.00	15,800
Walls	65	cu yd	720.00	46,800
Overflow trough (suspended)	75	cu yd	820.00	61,500
Suspended	45	cu yd	820.00	36,900
Actuator Supports	10	cu yd	820.00	8,200
Embedded accessories		Lump Sum		6,900
Equipment				
Mechanical				
Electric actuated gates				
6' X 6'	1	each	50,000.00	50,000
Electrical				
Electrical Labor & Materials		Lump Sum		10,000
Total - Gate & Spillway Structure				$3,727,000

Table 23.5-6 Opinion of Probable Construction Cost Northeast Gate Structure

BODR SUBMITTAL

EAA Reservoir A-1
New U.S. 27 Bridge Over Connecting Canal

OPINION OF
PROBABLE CONSTRUCTION COST
October 6, 2005

SUMMARY

General Requirements		614,000
Sitework		36,000
Bridge Structures		4,364,000
Project Reserve	5%	251,000
Contingencies	30%	1,580,000
Mid-Point of Construction		0
Rate = % 4%		
Time = Years 0		
TOTAL PROBABLE CONSTRUCTION COST		**$6,850,000**
ENGINEERING (Final Design and CPS)	0%	0
TOTAL PROBABLE PROJECT COST		**$6,850,000**

BLACK & VEATCH

New U.S. 27 Bridge Over Connecting Canal
Probable Construction Cost
October 6, 2005

Item Description	Quantity	Unit	Unit Cost $	Total Cost $
GENERAL REQUIREMENTS				
Mobilization		Lump Sum		73,600
Supervision		Lump Sum		230,000
Temporary facilities		Lump Sum		55,200
Temporary utilities		Lump Sum		36,800
Equipment rental & misc.		Lump Sum		18,400
Total - General Requirements				$414,000
SITEWORK				
Earthwork				
Site excavation				
Peat	0	cu yd	5.66	0
Caprock	0	cu yd	31.54	0
Finish grading	200	sq yd	1.21	200
Surfacings				
Concrete pavement	180	sq yd	66.00	11,900
Concrete curb & gutter	80	lin ft	27.50	2,200
Marking & signage		Lump Sum		22,000
Total - Sitework				$36,000
BRIDGE STRUCTURES				
Earthwork				
Structural excavation				
Excavation and Backfill	415	cu yd	22.00	9,100
Concrete, cast in place				
Abutment footing	95	cu yd	317.90	30,100
Abutment walls	237	cu yd	467.50	110,800
Parapet	187	cu yd	643.00	120,100
Deck Concrete	19,200	sq ft	22.00	422,400
Embedded accessories		Lump Sum		34,200
Concrete, precast				
Double-tee bridge	19,200	sq ft	23.43	449,900
Metal				
Deck steel	19,200	sq ft	155.20	2,980,000
Bridge railing	1,920	lin ft	148.50	285,100
Miscellaneous		Lump Sum		103,400
Total - Bridge Structures				$4,564,000

SECTION 24

EMERGENCY ACTION PLAN

24. EMERGENCY ACTION PLAN

An Emergency Action Plan (EAP) is commonly defined as a plan developed by a property owner that establishes procedures for notification to state and federal agencies, public off-site authorities, and other agencies of emergency actions to be taken in an impending or actual failure of a High Hazard impoundment. Agencies with EAP guidance include the Federal Energy Regulatory Commission (FERC), United States Bureau of Reclamation (USBR), State Dam Safety Offices, as well as local community/county representatives. Impoundments designated as High Hazard typically require the most stringent and detailed Emergency Action Plans. The impoundment breach modeling preformed on the EAA Reservoir A-1 shows that it is a High Hazard (also discussed in Section 5.2). U.S. 27 will be significantly impacted in the event of a breach, which will lead to life threatening conditions for motorists and impede emergency evacuation routes for southern Florida. The EAA Reservoir A-1 will need a comprehensive EAP that reflects its classification as a High Hazard impoundment.